CRANFIELD UNIVERSITY

LEDICIA PEREIRA GÓMEZ

VERTICAL FLOW CONSTRUCTED WETLANDS FOR TREATING UNSCREENED SEWAGE IN THE UK

SCHOOL OF WATER, ENERGY AND ENVIRONMENT
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MSc BY RESEARCH
Academic Year: 2015 - 2016

Supervisor: Dr. Yadira Bajón Fernández & Prof. Bruce Jefferson
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This thesis is submitted in partial fulfilment of the requirements for the degree of MSc BY RESEARCH

EXECUTIVE SUMMARY

The use of two stage vertical flow constructed wetlands (VFCWs) for sewage treatment post coarse screening is an established option in France.

The need to reduce the energy and maintenance requirements associated with small sewage works remains a key objective to the UK water industry. Two stage VFCWs have been identified as a candidate technology to meet these aspirations.

However, there is a paucity of information concerning operation and performance during the start-up period which could last up to two years as well as knowledge transfer relating to differences in hydraulic and organic loading patterns. Accordingly, the UK's first two stage VFCWs for municipal sewage treatment has been recently built and operated to assess its suitability.

Overall, the site performed similar to values reported in the literature regarding total suspended solids, biological demand and ammonium-N being respectively 6.2 ± 3.4 mg·L^{-1}, 5.6 ± 2.6 mg·L^{-1} and 5.8 ± 3.8 mg·L^{-1} compared to literature values of 10 ± 10 mg·L^{-1}, 6 ± 4 mg·L^{-1} and 5 ± 6 mg·L^{-1}, based on composite sampling. However, a key difference compared to operating systems in France was sustained operating periods beyond the design hydraulic load leading to long periods of surface ponding. This had two major impacts: a limiting ability to re-oxygenate the filter body affecting the nitrification performance and retardation of the sludge mineralisation rate reducing the operating infiltration rate and hydraulics of the filters. This highlights the hydraulic limitations of the young filter (5 months of operation) especially in winter conditions.

Future work has been suggested in order to adapt the technology to UK conditions such as extending first stage, optimising feeding strategy, using a storm and first stage overflow constructed wetland, aeration of the second stage or design based on infiltration rate.

Keywords:

Young filter, sludge layer, nitrification, hydraulics, ponding.

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I would like to start expressing my gratitude to my supervisors. Yadira Bajón Fernández for guiding me throughout the process and encouraging me until the end, making me see the positive things of this "experience" (you will understand), transmitting her always positive approach and challenging attitude, and also for correcting my pieces of writing about a thousand times. Bruce Jefferson for providing his expertise and always interesting and different point of view of the problems, allowing me to take a different approach in my scientific issues, and also for the final push. Finally I would like to thank Gaby Dotro for sharing her infinite knowledge about reed beds, even in busy times for her, and a big thank you for starting this project in the UK.

Secondly I would like to thank Severn Trent Water and the R&D team, in particular Ali Hull and Pete Vale, for putting resources into innovative projects like this which will allow to create a better environment for everyone by using eco-friendly technologies, and at the same time a technology that can help to achieve the main company targets. Also I want to thank the Asset Creation and Process Design Team, Dan Cunliffe, Andrew Richards, Ed Ruswa and Alexandra Cook, for dealing with design and commissioning issues and supporting me with technical data. I am sure the team will take this project to a successful stage overcoming difficult times.

I can't forget about the operational team on site during the commissioning and sampling stage: site manager Ady Woods and operators from MWH, always attending my requests with a smile in their faces, and also I want to give a special thank you to the Severn Trent Water Process Advisor, Tom Jordan, for the technical support and help with sampling material and procedures.

Lastly but not less important, a big thank you to my Cranfield colleague Olga Murujew, probably a key support in early stages showing me the lab facilities and other Severn Trent Water internal procedures, and also a key support in the hard times of writing, being by my side. Thank you for sharing lab hours and for all the work you did for the lab move. Wish you the best in your PhD, I am sure you will

become a Phosphorus Removal eminence and that you will find your way in the Water Industry.

I need to express a general thank you for this whole year, thanks to this project I have been able to put my career where I always wanted it to be, in the Wastewater Treatment Industry, this is already a success to me.

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LIST OF ABBREVIATIONS

BOD_5	Biological Oxygen Demand (5 days)
COD	Chemical Oxygen Demand
CW	Constructed Wetland
DEFRA	Department of Environment, Food and Rural Affairs
DM	Dry Matter
DO	Dissolved oxygen
DWF	Dry Weather Flow
F	Filter
FWS	Free Water Surface
HFCW	Horizontal Flow Constructed Wetland
HL	Hydraulic Load
IR	Infiltration Rate
N	Nitrogen
N_2	Nitrogen gas
N_2O	Nitrous Oxide
NH_4^+-N	Ammonium - Nitrogen
NO_3^--N	Nitrate - Nitrogen
OL	Organic Load
OM	Organic Matter
p.e.	Population equivalent
RB	Reed Bed
RI	Robustness Index
SSF	SubSurface Flow
TKN	Total Kjeldahl Nitrogen
TN	Total Nitrogen
TP	Total Phosphorus
TSS	Total Suspended Solids
UK	United Kingdom
VFCW	Vertical Flow Constructed Wetland
VS	Volatile solids
WWTP	Waste Water Treatment Plant

1 INTRODUCTION

The wastewater collection system in the United Kingdom (UK) is linked to around 9,000 waste water treatment plants (WWTPs), from which approximately 7,100 serve small agglomerations of less than 2,000 population equivalent (p.e.) (DEFRA 2012), representing an important part of the WWTPs network.

Traditional sewage treatment systems for small agglomerations are based on sedimentation and biological treatment with either trickling filters or rotating biological contactors. Whilst such approaches are effective and exert relatively low energy demands on a volume treated basis (0.1-0.3 $cKWh.m^{-3}$) regular maintenance and sludge collection is required. The latter results in the need for civil infrastructure to accommodate regular tanker visits (every 1-3 months). In addition, enhanced treatment is being required to meet stricter compliance requirements, especially related to ammonia and to provide sufficient capacity to respond to population growth. Accordingly, consideration is being given to identification of appropriate technologies for the future with the preferred aspiration of meeting the regulatory requirements whilst also reducing maintenance and sludge disposal.

Constructed wetlands (CWs) offer an attractive option to meet such requirements owing to the lower costs, reduced operation and maintenance requirements typically reported with the technology compared to traditional alternatives (Wu et al. 2015; Butterworth et al. 2016). Constructed wetlands (CWs) are engineered systems that have been designed and constructed to utilize the natural processes involving wetland vegetation, soils, and the associated microbial assemblages to assist in treating wastewaters (Vymazal 2011). Application for wastewater treatment is predominately based on subsurface systems where the water flows through a bed of planted media, oriented as horizontal flow (HF) or vertical flow (VF). Although CWs present the mentioned advantages, the footprint necessary for implementing this technology may be the most limiting factor for a broader application, especially where land resources are limited and population density is high (Wu et al. 2015).

In HFCWs, the wastewater is fed in at the inlet and continuously flows slowly through the bed before being collected via a water level control structure (Vymazal 2011). Most of the bed is anoxic/anaerobic due to permanent saturation of the beds such that HFCWs are very effective in removal of organics, suspended solids, and heavy metals but offer limited removal of ammonia-N (NH_4^+-N) or phosphorus (Vymazal 2011). Adaption of the technology can enable nitrification through forced aeration (Butterworth et al. 2013) or phosphorus removal by the use of reactive media (Vymazal 2011).

In VFCWs the wastewater is fed intermittently in batches which floods the surface layer. Wastewater then percolates through the bed and is collected by a drainage network at the bottom. Between feeding batches the bed drains completely which allows air to fill the void spaces within the bed. Accordingly, VFCWs operate predominately in aerobic environments enabling good nitrification to accompany removal of solids and organics (Vymazal 2011; Butterworth et al. 2016). In addition, multiple beds are used in parallel so that each bed can be rested to enable conditioning of the accumulated solids. Typical operating cycles are 7 days on and 7 days rest.

One specific embodiment of the technology is a two stage system which was developed in France by Cemagref more than 20 years ago (Lienard 1987) and was first applied by the SINT Company during the 1990s. The system comprises a coarse mechanical screen followed by two stages of vertical flow wetlands (Troesch et al. 2014; Paing and Voisin 2005) (Table 1-1, Figure 2-1). The first stage removes total suspended solids (TSS) and biological oxygen demand (BOD_5) while the second stage is designed for nitrification and polishing purposes. The success of the systems is dependent on maintaining aerobic conditions within the bed which is related to the hydraulic acceptance (hydraulic conductivity) of the filter that is linked to the feeding and resting management strategy adopted.

As the system operates, sludge accumulates on the top of the media bed. The hydraulic acceptance is therefore related to the hydraulic conductivity of this layer. Over time the sludge layer mineralises through aerobic metabolism that occurs during the rest cycles of the bed maintaining the hydraulic acceptance of the system. However, the activity and stabilisation of the sludge layer is reported to take significant time with previous studies reporting that approximately two years are needed before filters can be considered completely mature (Chazarenc and Merlin 2005; Paing et al. 2015).

An estimated 3,000 CWs treating raw sewage existed in France in 2014 (Troesch et al. 2014), for capacities of 20 to 6,000 population equivalent (p.e.) with reported de-sludge frequencies of 10-15 years (Molle et al. 2005). Typical effluent quality at the second-stage outlet are 10 ± 10 $mg \cdot L^{-1}$, 6 ± 4 $mg \cdot L^{-1}$ and 5 ± 6 $mg \cdot L^{-1}$ for TSS, BOD_5 and NH_4^+-N respectively based on composite sampling (Paing et al. 2015).

The main challenges related to wide spread application of two stage VFCWs are the need to decrease the land utilization requirements and the lack of optimization for total nitrogen (TN) removal (Prigent et al. 2013b). The two stage VFCWs standard design requires for the whole installation 3 to 5 $m^2 \cdot p.e.^{-1}$ (Troesch et al. 2014) which can limited application due to land availability. In addition, availability of a suitable sand substrate for the second stage beds can be challenging such that the standard French design is not always economically competitive (Troesch et al. 2014).

A compact system has been suggested in order to improve the footprint requirements of a two stages VFCWs. This system consists in just one stage of VFCWs with deeper filters and ventilations pipes at two different levels within the filter depth in order to facilitate the required oxygen transfer (Troesch et al. 2014). The reported quality of treated wastewater is 14 ± 7 $mg \cdot L^{-1}$, 19 ± 13 $mg \cdot L^{-1}$ and 14 ± 14 $mg \cdot L^{-1}$ for TSS, BOD_5 and NH_4^+-N respectively and hence is poorer than that achieved with standard designs (Paing et al. 2015). However, nitrification can be improved through recirculation (Prigent et al. 2013b) and total TN removal by inclusion of a saturated zone at the bottom of the filter (Troesch et al. 2014)

In terms of TN removal, the two stage VFCWs system is characterized by a high nitrification rate with 90% Total Kjeldahl Nitrogen (TKN) removal across the whole process congruent with maintaining aerobic conditions. Improvements for TN removal have been suggested in terms of the use of hybrid combinations (Molle, Prost-boucle, and Lienard 2008) (VF followed by HF), the implementation of a saturated layer in the bottom of the filters (Silveira et al. 2015) and recirculation (Prost-Boucle et al. 2012; Prigent and Paing, et al. 2013).

Table 1-1 Summary standard design and operational parameters found in literature

Design feature	French design	
HL ($m.d^{-1}$)	$0.37\ m{\cdot}d^{-1}$ (Morvannou et al. 2015, Troesch et al. 2014)	
Load ($g{\cdot}m^{-2}{\cdot}day^{-1}$)	(Morvannou et al. 2015)	
- **TSS**	150	
- **COD**	300	
- **TKN**	25-35	
Area ($m^2{\cdot}p.e.^{-1}$)	(Morvannou et al. 2015, Troesch et al. 2014)	
- **Total**	2	
- **First stage**	1.2	
- **Second Stage**	0.8	
Media	(Paing et al 2015)	(Troesch et al. 2014)
- **First stage**	40–50 cm gravel 2-8 mm 15–20 cm gravel 10-20 mm 20 cm gravel 20-40 mm	>30 cm gravel 2-8 mm 10–20 cm gravel 5-20 mm 10-20 cm gravel 20-40 mm
- **Second stage**	40 cm sand 0-4 mm 15–20 cm gravel 4-10 or 4-20mm 20 cm gravel 10-20 or 20-40 mm	>30 cm sand $0.25<d_{10}<0.4$ mm 10–20 cm gravel 3-10 mm 10-20 cm gravel 20-40 mm
Feeding system		
- **First stage**	Screening Siphon/pumping station	
- **Second stage**	Siphon/pumping station	

Feeding points - **First stage** - **Second stage**	1 per max 50 m^2 (Troesch et al. 2014) 1 per 1 m^2 (Troesch et al. 2014
Feeding/resting period - **First stage** - **Second stage**	- 3.5 days / 7 days (Morvannou et al. 2015, Troesch et al. 2014) - 1 week / 2 weeks (Paing et al. 2015) - 1 week /1 week (Paing et al. 2015) - 3.5 days / 3.5 days (Morvannou et al. 2015)
Volume per batch (cm over surface) - **First stage** - **Second stage**	- 3-4 cm over surface(Paing 2015) - 1.3 cm (Molle 2014) - 3 cm (Paing 2005) - 4cm (Paing 2005)
Sludge accumulation (cm·year^{-1})	- 1.5 (Molle et al. 2005) - 2.5 (Molle 2014) - 1-2 (Troesch et al. 2014)
Reed type - **Type** - **Density**	Phragmites australis 1 every 50 cm in each direction (Boutin and Liénard 2003)

In the UK CWs started to be accepted as a conventional technology for wastewater treatment during 1990s, when the number of CWs installed started to increase (Cooper 2007), with approximately 670 sites operating for municipal sewage and a further 266 for private treatment systems (Constructed Wetlands Association (CWA) database 2011). The vast majority of CWs are installed for municipal tertiary treatment in the form of sub surface horizontal flow beds (Butterworth et al. 2013). Recent adaptions have seen a rise in the number incorporating forced aeration with a recent report identifying around 50 sites (CWA 2014). The majority of other applications are utilised on private treatment facilities such as secondary sewage treatment, storm sewage overflow treatment and treatment of waste water with a different profile than sewage: mine water, industrial, landfill leachate, agricultural runoff, surface runoff and road runoff (Cooper 2007; CWA database 2011).

The need to reduce the energy and maintenance requirements associated with small sewage works remains a key objective to the UK water industry. A number of existing sites are reaching their operational life with the need for adaption or replacement to meet tightening discharge consents and/or increased capacity due to population growth. Two stage (French design) VFCWs have been identified as a candidate technology to meet these aspirations.

However, there is a paucity of information concerning operation and performance during the start-up period which could last up to two years as well as knowledge transfer relating to differences in hydraulic and organic loading patterns. Accordingly, the UKs first two stage VFCWs for municipal sewage treatment has been recently built and operated to assess its suitability. In this scheme adaption was included to negate screening to reduce maintenance of the overall site.

1.1 Aim and objectives

The overall aim of the thesis is to understand the efficacy of a two stage VFCWs for complete municipal treatment in the UK during its initial start-up period. Embedded within the overall aim is an ambition to understand the required adaptations needed for effective use in the UK and the risks associated with implementing full scale systems without prior pilot testing in local conditions. The full scale investigation will serve as a base to assess the potential for widespread implementation of the technology for small wastewater treatment works.

To deliver against this research aim the following specific objectives were defined:

1. Establish the robustness of the scheme during the commissioning stage for pollutant removal, nitrification start-up time and compliance with UK legislation based on spot sampling.

2. Investigate the link between the characteristics of the accumulated sludge and the hydraulic acceptance of the first stage.

3. Establish the appropriateness of the technology to meet UK requirements and highlight key design and commissioning translations required when implementation the technology in the UK.

1.2 Thesis plan

This is an industrial based project. The thesis is organised by chapters. Chapter 1 introduces the background of the project as well as defining the project aims and objectives, and develops a literature review including the state of art of the technology. Chapter 2 presents the findings from the investigation of the initial operating period of a full scale two stage VFCWs for unscreened municipal sewage treatment. All chapters have been written by Ledicia Pereira and edited by Professor Bruce Jefferson and Dr. Yadira Bajón Fernández. All laboratory work and sampling was undertaken by Ledicia Pereira with exception to some analysis of Chapter 2 (TSS, BOD, Ammonia and Chemical Oxygen Demand (COD)), for which samples were sent to an external contractor and to support for full-scale sampling by Thomas Jordan (Severn Trent Water process advisor). Peter Vale, Daniel Cunlinfe, Eddison Ruswa and Alexandra Cooke contributed as members of Severn Trent Water team providing technical advice and process information.

Aspects of the work have been presented at two conferences:

- Pereira Gomez L., Vale P., Ruswa E., Cunliffe D., Bajón Fernández Y., Dotro G. and Jefferson B. 2015 "Treatment of unscreened domestic wastewater with vertical flow wetlands". *WETPOL 2015 - 6th International Symposium on Wetland Pollutant Dynamics and Control. Annual Conference of the Constructed Wetland Association (14th-18th of September 2015).* Poster presentation.
- Pereira Gomez L., Vale P., Ruswa E., Cunliffe D., Bajón Fernández Y., Dotro G. and Jefferson B. 2015 "Treatment of unscreened domestic wastewater with vertical flow wetlands". *Low Energy Wastewater Treatment Systems conference at Cranfield University* 2015. Poster presentation.

2 TWO STAGE VERTICAL FLOW CONSTRUCTED WETLANDS FOR SEWAGE TREATMENT: EARLY STAGE OPERATION AND PERFORMANCE WITHOUT SCREENING.

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2.1 Abstract

Two stage vertical flow constructed wetlands (VFCWs) for sewage treatment post coarse screening is an established option in France. However, reported information concerning operation and performance during the start-up period as well as knowledge transfer relating to differences in hydraulic and organic loading patterns is limited. Accordingly, the UKs first two stage VFCWs for municipal sewage treatment has been recently built and operated to assess its suitability.

Overall, the site performed similar to values reported in literature regarding total suspended solids, biological demand and ammonium-N being respectively 6.2 ± 3.4 $mg{\cdot}L^{-1}$, 5.6 ± 2.6 $mg{\cdot}L^{-1}$ and 5.8 ± 3.8 $mg{\cdot}L^{-1}$ compared to literature values of 10 ± 10 $mg{\cdot}L^{-1}$, 6 ± 4 $mg{\cdot}L^{-1}$ and 5 ± 6 $mg{\cdot}L^{-1}$, based on composite sampling. However, a key difference compared to operating systems in France was sustained operating periods beyond the design hydraulic load leading to long periods of surface ponding. This had two major impacts: a limiting ability to re-oxygenate the filter body affecting the nitrification performance and retardation of the sludge mineralisation rate reducing the operating infiltration rate and hydraulics of the filters.

The implications for further use of the technology are a need to ensure appropriate adaption linked to the actual hydraulic loading profile observed at the target site.

2.2 Introduction

The use of two stage vertical flow constructed wetlands (VFCWs) for sewage treatment post coarse screening is an established option in France with an estimated 3,000 CWs in 2014 for capacities of 20 to 6,000 population equivalent (p.e.) (Troesch et al. 2014). The systems operate on a fed batch basis with typically three beds in the first stage and two beds in the second (Figure 2-1). The beds are also operated on a feed/rest cycle of 3.5 days on / 7 days off for the first stage and 7 days on / 7 days off for the second. The combination of operating cycles enables the systems to remain aerobic and for the accumulated sludge to be conditioned and mineralised enabling long term operation without the need for desludging with typical reported de-sludge frequencies of 10-15 years (Molle et al. 2005). The aerobic conditions facilitate nitrification with typical mean effluent qualities (based on 24 hour composites) reported across a number of sites of 10 ± 10 mg·L^{-1}, 6 ± 4 mg·L^{-1} and 5 ± 6 mg·L^{-1} for TSS, BOD_5 and NH_4^+-N respectively (Paing et al. 2015).

The efficacy of the process is dependent on maintaining aerobic conditions that requires sufficient time between fed cycles for sufficient oxygen to diffuse into the biofilm on the media. To ensure this it is recommended that the beds are flooded for a maximum 15.5 hours per day (Arias López 2013) such that the recommended design hydraulic loading (HL) rate is 0.37 m.d^{-1} (Morvannou et al. 2015; Troesch et al. 2014) corresponding to area requirements of between 2-2.5 m^2/p.e. (Molle et al. 2005). Solids accumulation predominately occurs in the first stage with the second stage delivering nitrification and polishing of organics and solids (Troesch et al. 2014). Accordingly, the conditioning of the accumulated sludge in the first stage strongly influences the overall hydraulic acceptance of the systems (Beach et al. 2005). The sludge layer conditions and mineralises during the rest cycles with establishment of mature systems reported to take up to two years (Chazarenc and Merlin 2005; Paing et al. 2015). However, there is a paucity of data on the operation and performance of the system in these initial years and during winter seasons when the activity of the accumulated sludge layer is expected to be reduced. In particular the impact of the accumulated sludge on hydraulic acceptance is unclear, especially in relation to the large natural variations in hydraulic loading that occurs at small works due to impact of storm water which can lead to peak flows more than 6 times the dry weather flow of the system.

Consequently, the aim of the paper is to understand the operation and performance of a two stage vertical flow wetland system during the initial years of operation. To achieve this a full scale system recently installed in the UK to treat the full flow of a sewage catchment has been monitored in terms of operation and performance to establish key challenges that need to be considered when implementing the technology.

2.3 Materials and Methods

2.3.1 Description and operation of a full-scale VFCWs.

The site treats a population equivalent (p.e.) of 941 with discharge consents of 50, 30 and 15 $mg \cdot L^{-1}$ for TSS, BOD_5 and NH_4^+-N respectively in terms of spot sampling. The consented dry weather flow (DWF) was 2.27 $L \cdot s^{-1}$ and 6 DWF of 11 $L \cdot s^{-1.}$ Such that the corresponding hydraulic loading rate for DWF was 0.52 $m.d^{-1}$ compared to the typical design value for this technology of 0.37 $m.d^{-1}$. The constructed wetlands comprised of three first stage filters with an active area of 1.2 m^2 $p.e.^{-1}$ followed by two second stage filters sized at 0.8 m^2 $p.e.^{-1}$ (Figure 2-1,Table 2-1) The beds were planted with *Phragmites australis* at a density of 4 per m^2. The first stage filters (BOD_5 and TSS removal) (Figure 2-2 (a)) were operated on a 3.5 days on / 7 days rest cycle. Each fed batch consisted of 7.5 m^3 of un-screened sewage distributed through eight feeding pipes (1 per 47 m^2). The second stage filters (nitrification and BOD_5/TSS polishing) (Figure2-2)(b)) were gravity fed in batches by means of a syphon onto a series of perforated pipes located along the length of the filters with feeding points in a density of 1 per m^2. The second stage filters were operated on a 7 days on / 7 days off basis.

The VFCWs were commissioned in two phases to manage the transition from the existing works. In stage one, mid-July to mid-September, the first stage operated with two filter beds and the flow was limited to 5 $L \cdot s^{-1}$. In phase two, the third filter bed was started and after one month of operation the full flow to the site was passed through the VFCWs (up to a maximum of 11 L s^{-1}). During the commissioning period the site needed to be compliant with discharge limits, utilizing a tertiary bio-filter as buffer solution until the new scheme performed reliably.

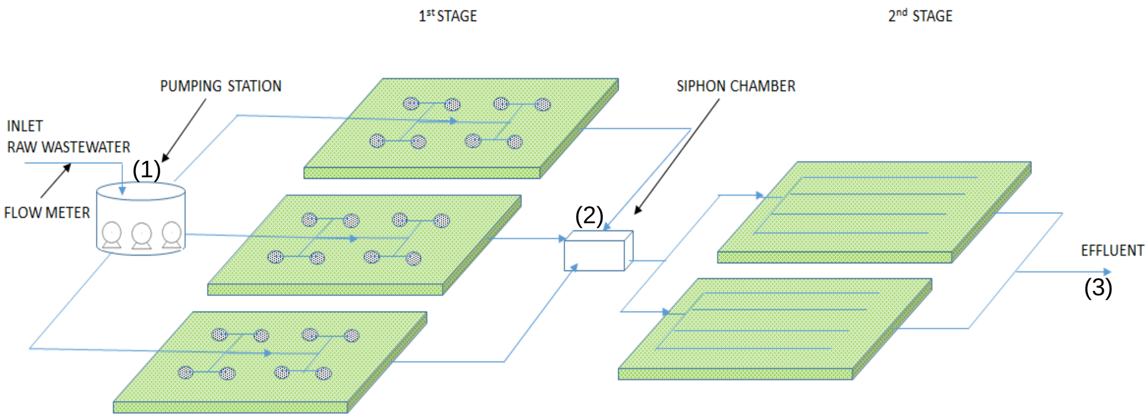


Figure 2-1 Flowsheet two stages VFCWs and sampling points (1) (2) (3)

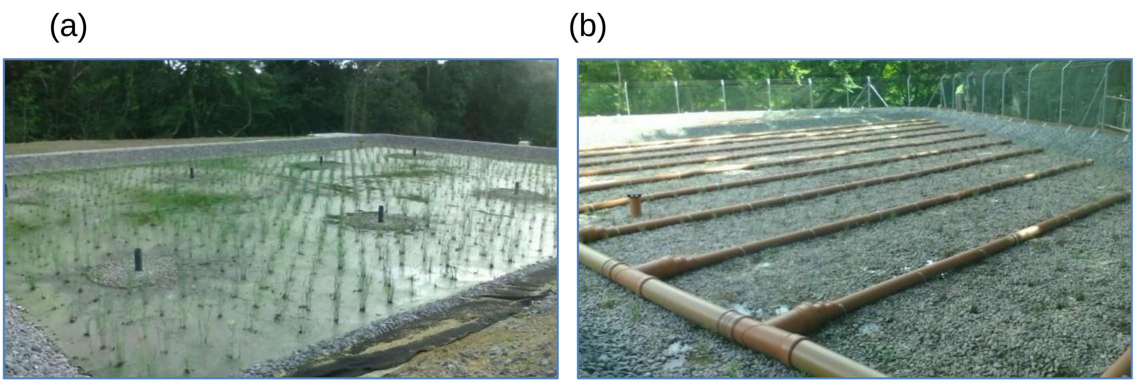


Figure 2-2 Filter first stage (a) and second stage (b)

Table 2-1 Design features for the scheme

Design feature	1st stage		2nd stage	
Surface per p.e	1.2 $m^2 \cdot p.e.^{-1}$		0.8 $m^2 \cdot p.e.^{-1}$	
Number of filters	3		2	
Type of reeds, density	*Phragmites australis* 4 per m^2			
Feeding system	No screening, pumping station		Syphoning	
Feeding density	Fountain 1 per 47 m^2		Perforated pipes 1 feeding point per m^2	
Feeding/resting cycles	3.5 days / 7 days		7 days / 7 days	
Filter media	**Depth**	**Particle size**	**Depth**	**Particle size**
	40 cm	gravel 2-6mm	40 cm	sand 0-4 mm
	20-40 cm*	gravel 10-63 mm*	40 cm	gravel 2-6mm
			20cm	gravel 20-40mm
			0-30cm*	gravel 10-63mm*

* Draining layer

2.3.2 Site monitoring for different pollutants.

Wastewater samples were collected up to 3 times a week for 24 hour composite sampling using an auto-sampler (Aquacell P2-compact CL-1010, Aquamatic, UK) and up to 5 days a week for spot sampling during the first 5 months of operation of the scheme (mid-July to mid-December). The sampling points were: inlet (1), effluent first stage (2) and effluent second stage (3) (Figure 2-1) for 24 hour composite samples and effluent second stage (3) for spot samples. Samples were analysed for TSS, NH_4^+-N, nitrates (NO_3^-–N), BOD_5, chemical oxygen demand (COD), total nitrogen (TN) and total phosphorus (TP), using analysis methods in accordance with British standards (SCA bluebook 236, 2011).

A robustness index (RI) was calculated against a treatment goal (Tgoal) and the percentage time spent under the treatment goal (Equation 2-1) (Hartshorn et al. 2015) based on 24 hour composite samples.

$$\boldsymbol{RI} = \left[\left(\mathbf{1} - \frac{\boldsymbol{G\%}}{\mathbf{100}}\right) \times \frac{\boldsymbol{T_{90}}}{\boldsymbol{T_{50}}}\right] \times \left[\frac{\boldsymbol{T_{50}}}{\boldsymbol{T_{goal}}} \times \frac{\boldsymbol{G\%}}{\mathbf{100}}\right] \qquad \textbf{Equation 2-1}$$

where G% is the percentile value below treatment goal, T_{90} is the treatment achieved at 90th percentile and T_{50} is the treatment achieved at 90th percentile.

The first term of Equation 2-1 represents uniformity, while the second represents overall filter performance against the goal. A RI value close to 1 indicates a more robust process (Hartshorn et al. 2015).

Sampling results were grouped by hydraulic loading rate as close to design value (<0.4 $m \cdot d^{-1}$), medium HLs (0.4 to 0.6 $m \cdot d^{-1}$), and high HLs (≥ 0.6 $m \cdot d^{-1}$). Similarly, data were grouped by organic load (OL) with groups <75, 75 to 150 and ≥150 g $BOD_5 \cdot m^2 \cdot d^{-1}$ for the first stage CW and <15, 15 to 30 and ≥ 30 g $BOD_5 \cdot m^2 \cdot d^{-1}$ for the second stage CW. Comparison between groups depending on HL and OL were analysed using a sigma-restricted parametrization (ANOVA) at an interval of confidence $p < 0.05$.

2.3.3 Hydraulic behaviour and sludge characteristics in the first stage.

Infiltration rate (IR) and sludge layer characteristics were measured in month 5-6 of operation (December-January) and month 8 (March). Infiltration rate (IR) was measured by water ponding level differences in the surface on the filters and elapsed times between batches, using two pressure probes (Levelogger Edge, Solinst, Canada) located in different points on the filter surface. Measurements were performed during feeding periods for filters number 1 and 3 in the two campaigns.

Sludge samples from three different points in each filter (F1 and F3) were collected in each of the two sampling campaigns. Each sample was 5 cm x 5 cm in area and comprised of all the accumulated height. Each sample was homogenised and measured for dry matter content (DM) and organic matter (OM) content in terms of volatiles solids (VS) according to British standards (SCA bluebook 236, 2011). Accumulated height was also recorded. Daily HL values were calculated using a flow meter (ultrasonic level transducer DB Mach 3, Pulsar, UK) located in the inlet channel, ahead of the pumping station. This HL were used to calculate daily pollutant mass loads.

2.4 Results and discussion

2.4.1 Treatment performance

Overall treatment across the system, excluding periods of storm events (HL> 0.6 $m \cdot d^{-1}$) generated average residual levels on a 24 hour composite basis of 6.2 ± 3.4 $mg \cdot L^{-1}$, 5.6 ± 2.6 $mg \cdot L^{-1}$ and 5.8 ± 3.8 $mg \cdot L^{-1}$ for TSS, BOD_5 and NH_4^+-N (excluding 15 days of nitrification start-up time) respectively (Table 2-2). Such treatment is commensurate with reported levels across sites in France which are reported to be 10 ± 10 $mg \cdot L^{-1}$, 6 ± 4 $mg \cdot L^{-1}$ and 5 ± 6 $mg \cdot L^{-1}$ for TSS, BOD_5 and NH_4^+-N (Paing et al. 2015) confirming the efficacy of the technology.

When examining the two stages individually, the mean NH_4^+-N removal rate in the first stage was 6.2 $g \cdot m^{-2} \cdot d^{-1}$ with a corresponding nitrate production rate of 0.9 $g \cdot m^{-2} \cdot d^{-1}$ corresponding to a mean concentration of 1.6 ± 2 $mg \cdot L^{-1}$. Comparison to previously reported systems reveals such levels to be lower than expected at 11 to 15.5 $g \cdot m^{-2} \cdot d^{-1}$ (Morvannou et al. 2015) based on a 60 % removal of the Total Kjeldahl Nitrogen (TKN) load and a ratio NH_4^+-N/TKN of 0.74 Further analysis across the filter beds reveals a deficit in the mass balance of 5.2 $g \cdot m^{-2} \cdot d^{-1}$ of N which are accounted for in terms of either: (a) the mass of NO_3-N denitrified, (b) the mass incomplete denitrified with nitrous oxide production (N_2O) and (c) mass accumulated in the system.

Previous investigations have indicated that a reduction of about 10% of the incoming organic load onto a first stage filter can be attributed to heterotrophic denitrification providing nitrification is occurring in the first stage filters (Morvannou et al. 2014). Ammonia is removed through a two-step process whereby it first adsorbs into the biofilm during the fed cycles and then is nitrified during the rest period when oxygen is supplied through diffusion from the air filled pore spaces. Previous simulated estimates suggest that, during the feeding period, around 33% of the incoming N load can be removed through adsorption and 44% is nitrified (Morvannou et al. 2014). This compared to the total removal in the current study of 30 ± 19% for HL < 0.6 $m \cdot d^{-1}$ and combined with the low NO_3^--N production, suggests adsorption to be a major influence on the overall removal during the initial stages of operation. The observations in the current study are also influenced by the fact that excessive ponding occurred during the initial stages of operation where the guide level of a maximum of 14.5 hours a day ponding was substantially exceeded (see section 2.4.2.1). This would have prevented the supply of sufficient oxygen to the biofilm and thus inhibiting the establishment of an active nitrification activity. Continued removal of ammonia in the first stage suggest that the adsorption capacity of the system had not yet been exhausted.

Although mass removal of NH_4^+-N in the first stage was lower than the one reported in previous literature, the NH_4^+-N mean concentrations feeding the second stage were 28.1 ± 7.4 $mg \cdot L^{-1}$ which is in line with typical values reported in previous monitored sites in France (32 ± 17 $mg \cdot L^{-1}$) (Table 2-2). Effluent concentrations and removal efficiencies in the second stage were respectively 5.8 ± 3.8 $mg \cdot L^{-1}$ and 77.9 ± 13.4 % in agreement with the literature values (5 ± 6 $mg \cdot L^{-1}$ and 78 ± 18 % in TKN removal) (Table 2-2). The corresponding second stage mass removal rates were 8.2 $g \cdot m^{-2} \cdot d^{-1}$.

The second stage started to nitrify within 15 days of operation with a decrease in NH_4^+-N (Figure 2-3) matching an effluent release of nitrates (Figure 2-4). In relation to each cycle, peaks in ammonium were observed by the end of the feeding periods with corresponding nitrates peaks at the start of each cycle. This is consistent with the expected pathways of operation whereby the ammonium is accumulated/adsorbed in the biofilm during the feeding period, and it is then nitrified during the resting period and between batches (Paing et al. 2015) and nitrates flushed from the beds when the feed starts again (Boutin and Liénard 2003) (see patterns Figure 2-3 and Figure 2-4).

After 43 days of operation, surface ponding developed on the second stage filter with a corresponding increase in effluent NH_4^+-N (Figure 2-3). To illustrate, after the start up period the maximum ammonia concentration was 11.5 mg·L^{-1} which increased to 13.4 mg·L^{-1} after day 43 and exceeded target level on day 85 at which point the NH_4^+-N reached 17.2 mg·L^{-1} with a maximum value of 18.9 mg·L^{-1} on day 99. During this time the tertiary bio-filter was in place as a buffering solution for discharging in the environment safely. Amelioration was attempted in terms of alteration of the fed/rest cycle to 5 days on / 5 days rest (days 65 to 105 in Figure 2-3) but this did not improve performance. It was further hypothesised that ponding had occurred to surface clogging of the finer media used in the second stage. However, the average TSS load on the second stage was 28 g·m^{-2}·d^{-1} (Figure 2-5) with episodic values above 45 g TSS ·m^{-2}·d^{-1} which is below the previously reported rate of 45 g·m^{-2}·d^{-1} required to create permanent ponding with the media used (Langergraber et al. 2003). To mitigate against this a surface scrap was performed on day 108 with a corresponding reduction in effluent ammonia to a mean level of 2.8 mg.L^{-1}. However, this period of time corresponded to a period of storm events with an increased flow of 3 times DWF value which is sufficient to account for the reduced concertation due to dilution. High ponding rates also reappeared rapidly suggesting that surface solids deposition was not the main reason for the observed ponding rates.

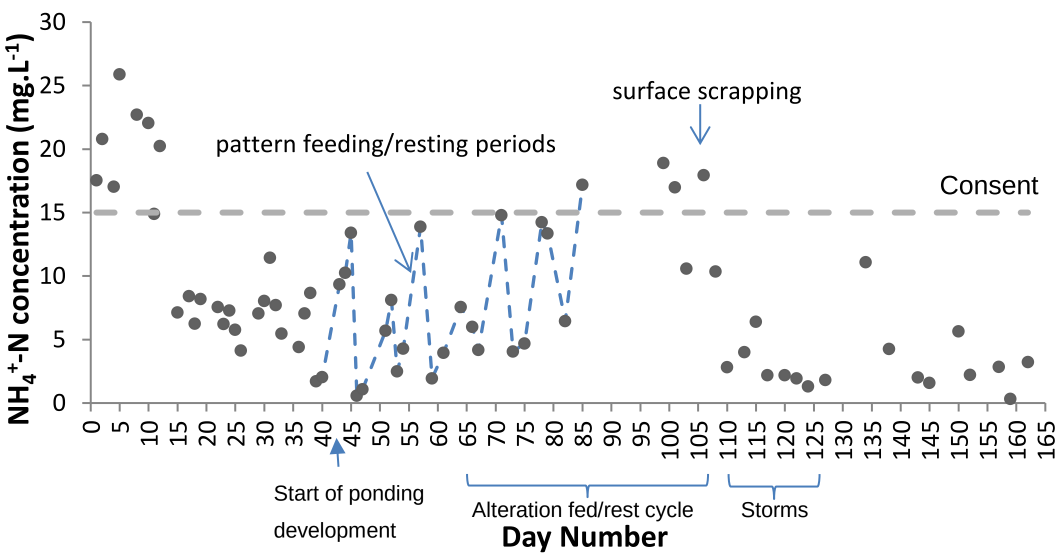


Figure 2-3 Evolution of NH_4^+-N spot samples over time in the effluent of second stage of a two stages VFCWs system

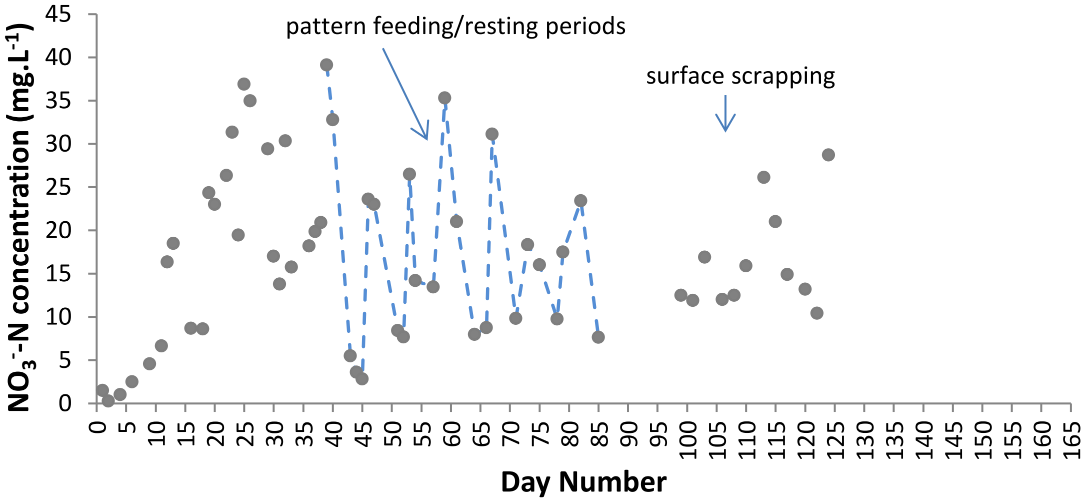


Figure 2-4 Evolution of NO_3^--N spot samples over time in the effluent of second stage of a two stages VFCWs system

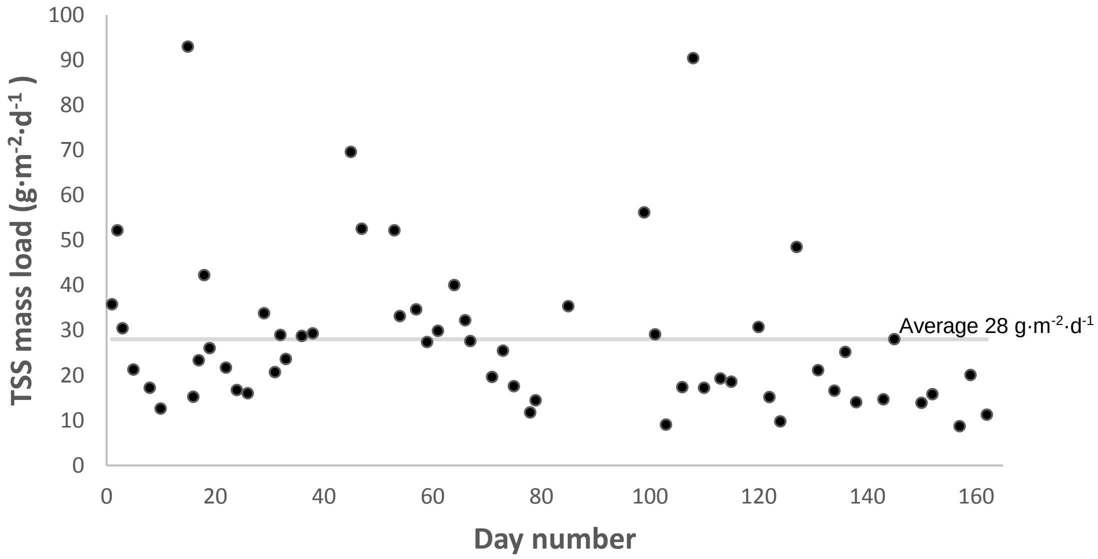


Figure 2-5 TSS mass load onto second stage of a two stages VFCWs system over time

Overall phosphorus removal was high at 94.8 ± 5.2 %, with a mean concentration of 0.33 ± 0.27 mg·L^{-1} in the effluent of the second stage commensurate with accumulation of P in the developing sludge layer and adsorption onto the fresh media during early periods of operation. This is in agreement with findings for other wetland systems in early periods of operation (Paing et al. 2015). Phosphorus removal is expected to significantly decrease with time as the media reaches saturation capacity (Paing et al. 2015; Vymazal 2010), and is expected to reach a level of 30% of the incoming levels once the system has stabilised (Paing et al. 2015).

The ammonia value after second stage of VFCW during warm months (July to October) was higher than the value recorded during colder months (November and December), being the means 6.4 ± 3.7 mg·L^{-1} and 3.5 ± 1.9 mg·L^{-1} of NH_4^+-N for the two periods respectively. Expectations were that ammonia is negatively affected during the colder months. The fact that the relation between ammonia performance and season is not clear aligns with findings in literature (Paing et al., 2015). However it is necessary to mention that in this case the hydraulic load pattern and batch management heavily influenced filter re-oxygenation and the ammonia performance as explained in section 2.4.2.1, and this is suggested to possibly minimise the temperature impact. In addition, during colder months storm events were more frequent and influent sewage was diluted, lowering the NH_4^+-N concentration in the effluent.

Table 2-2 Concentrations in 24 hour composite samples and removal efficiencies for the different stages of the first and second stages of the monitored UK two stages VFCWs (first 5 months of operation) compared to a survey based on 169 full scale systems in France (Paing et al. 2015) for HL < $0.6 m \cdot d^{-1}$

($mg \cdot L^{-1}$)			TSS	BOD_5	COD	NH_4^+-N	NO_3^--N	TN	TP	TSS	BOD_5	COD	TKN	NH_4^+-N	NO_3^--N	TN	TP
			UK system							**French systems**							
HL<$0.6 m \cdot d^{-1}$	Raw wastewater	Mean	315.5	260.4	665.9	43.1	0.81	64.4	8.7	353	360	841	94	70			12
		SD	146.4	97.1	236.6	12.8	0.47	15.4	3.3	207	159	340	27	22			4
		N	35	34	35	29	18	16	16								
	1st stage effluent	Mean	55.8	45.8	169.1	28.1	1.6	46.3	4.5	43	46	153	35	32	31		9
		SD	20.8	17.9	59.3	7.4	2	10.5	3.3	38	43	91	18	17	36		3
		N	35	32	36	32	31	18	19								
	2nd stage effluent	Mean	5.62	5.1	38.7	5.8	19.2	31.7	0.33	10	6	51	7	5	56		8
		SD	2.5	1.7	11.3	3.8	8	7.8	0.27	10	4	21	7	6	25		3
		N	34	32	33	29	30	15	16								
Efficiencies	1st stage effluent	Mean	79.2%	79.9%	73%	30.4%		26.8%	44.4%	85%	86%	80%	62%			40%	30%
		SD	11.4%	10%	9.9%	19.5%		12.3%	21.5%	15%	13%	11%	16%			33%	26%
		N	32	28	32	28		15	16								
	2nd stage effluent	Mean	88.4%	87.7%	74.7%	77.9%		28.6%	90.8%	53%	79%	59%	78%			-2%	-12%
		SD	6.7%	4.8%	6.3%	13.4%		12.4%	7.5%	78%	21%	28%	18%			56%	105%
		N	31	30	31	28		15	16								
	Global	Mean	98.1%	97.7%	93.5%	86.8%		47.2%	94.8%	96%	98%	93%	93%			39%	30%
		SD	1.1%	1.4%	2.8%	8%		13.2%	5.2%	4%	1%	4%	7%			30%	28%
		N	29	29	27	25		12	13								

Robustness analysis of the system based on 24 hour composite samples including storm events revealed the two stage system to be robust to suspended solids and BOD removal as indicated by the near vertical line and no tail on the curve (Figures 2-6, 2-7). Greater variability was observed for ammonia with a curve that is indicative of a system exhibiting limited robustness (Figure 2-8). Effluent data revealed that 95% of the results for the overall performance (effluent of the second stage), were below 9.4 $mg \cdot L^{-1}$ for TSS, 7.7 $mg \cdot L^{-1}$ for BOD_5 and 11.9 $mg \cdot L^{-1}$ for NH_4^+-N. The shape of these curves is similar to the robustness curves reported for other systems operated in France (Morvannou et al. 2015). However, the concentrations reported for 90% percentile in the previous study are higher at 25 $mg \cdot L^{-1}$ for TSS, 22 $mg \cdot L^{-1}$ for BOD_5 and 26 $mg \cdot L^{-1}$ for TKN. This is also different to the robustness curves for NH_4^+-N reported for tertiary aerated gravel-based systems, achieving 90% below 0.5 $mg \cdot L^{-1}$ and 95% below 3 $mg \cdot L^{-1}$ (Butterworth 2014).

The RI allows consideration of the impact of tightening consents through changing the target consent level (Butterworth 2014). A lower RI indicates a more robust process. Considering the value 1 as a robust system, 2 stage VFCW remained robust at target levels of 7.5 $mg \cdot L^{-1}$ of NH_4^+-N, 7 $mg \cdot L^{-1}$ of TSS and 6 $mg \cdot L^{-1}$ of BOD_5, while conventional treatment works followed by aerated CW remained robust down to 0.5 $mg \cdot L^{-1}$ of NH_4^+-N, based on spot sampling (Butterworth 2014) (Figure 2-9).

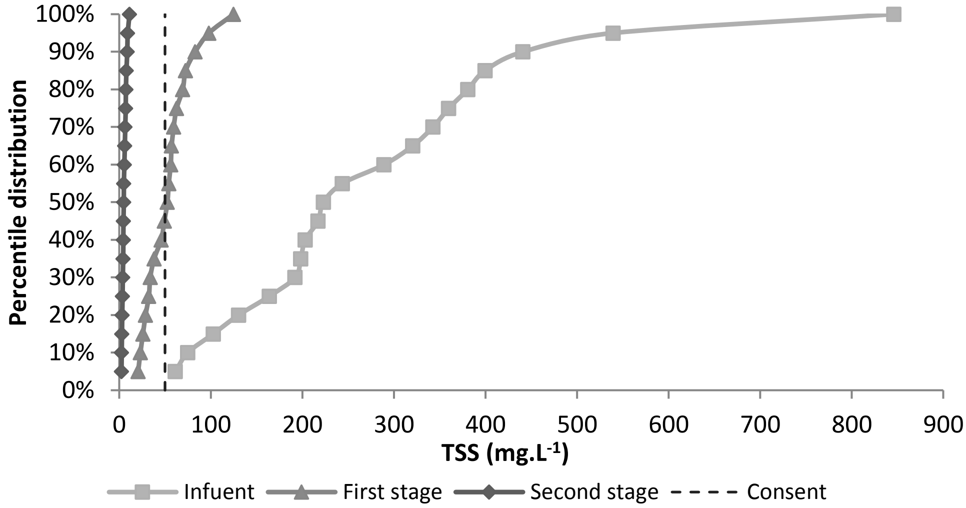


Figure 2-6 Percentile distribution of TSS 24h-composite samples in the inlet (N=56), effluent of the first stage (N=59) and effluent of the second stage (N=55) including all HL events.

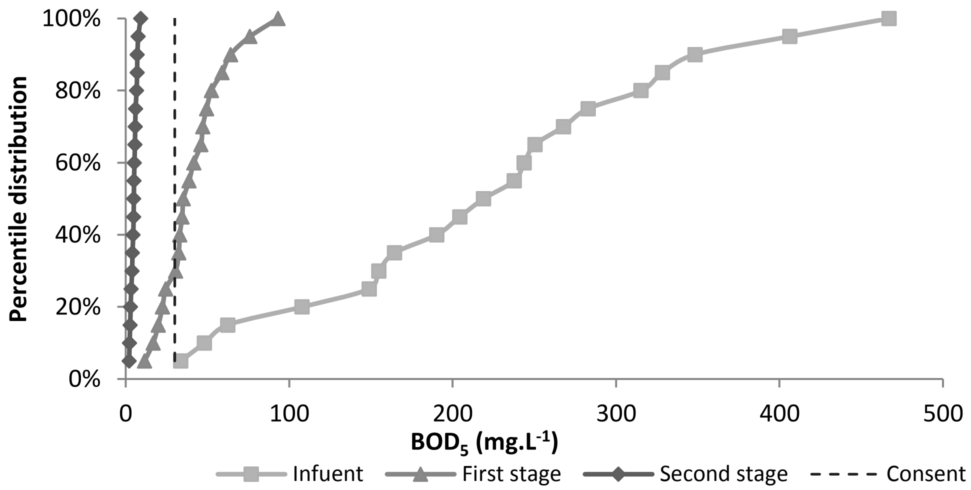


Figure 2-7 Percentile distribution of BOD_5 24h-composite samples in the inlet (N=55), effluent of the first stage (N=58) and effluent of the second stage (N=53) including all HL events

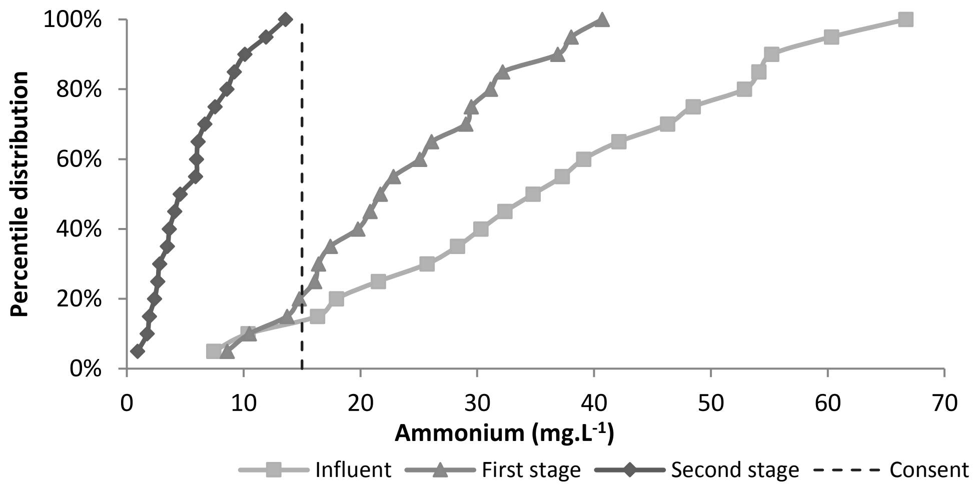


Figure 2-8 Percentile distribution of NH_4^+-N 24h-composite samples in the inlet (N=56), effluent of the first stage (N=55) and effluent of the second stage (N=49) including all HL events

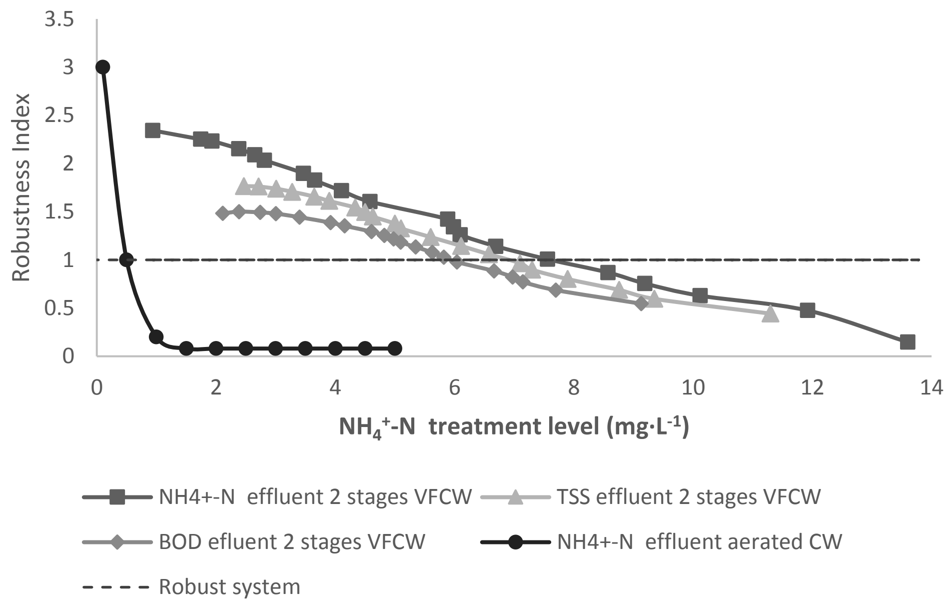


Figure 2-9 Robustness index (RI) of 2 stages VFCW for different pollutant treatment levels and compared to RI of a conventional works followed by aerated CW (adapted from Butterworth 2014).

Analysis of the performance in relation to hydraulic loading rate was based on grouping the data into three categories: close to design value (< 0.4 $m \cdot d^{-1}$), medium HLs (0.4 to 0.6 $m \cdot d^{-1}$) and high HLs corresponding to storm events (≥ 0.6 $m \cdot d^{-1}$). Increasing HL decreased the mean residual concentrations in the first stage filter (Figure 2-10 (a)). This can be explained by dilution of the incoming sewage during storm events (Paing et al. 2015). In contrast, no statistical difference (p value > 0.05 in the ANOVA analysis) was observed in the second stage (Figure 2-11 (a)) indicating it was able to buffer the variability of effluents from the first stage. Equivalent analysis based on OL with categories < 75, 75 to 150 and ≥ 150 g $BOD_5 \cdot m^{-2} \cdot d^{-1}$ for the first stage; and <15, 15 to 30 and ≥ 30 g $BOD_5 \cdot m^{-2} \cdot d^{-1}$ for the second stage; revealed no statistical difference (p value > 0.05 in the ANOVA analysis) across the stages for all pollutants (Figure 2-10 (b), Figure 2-11(b)). This is consistent with a previous trial which showed little correlation between OL and treatment performance (Paing et al. 2015) but it differs from other references reporting a reduced NH_4^+-N treatment performance with increasing OL in the first stage (Morvannou et al. 2015). In the current case, the absence of correlation between residual NH_4^+-N and OL applied in the first stage can be explained by the fact that the removal of NH_4^+-N in the first stage is linked to adsorptive uptake rather than nitrification.

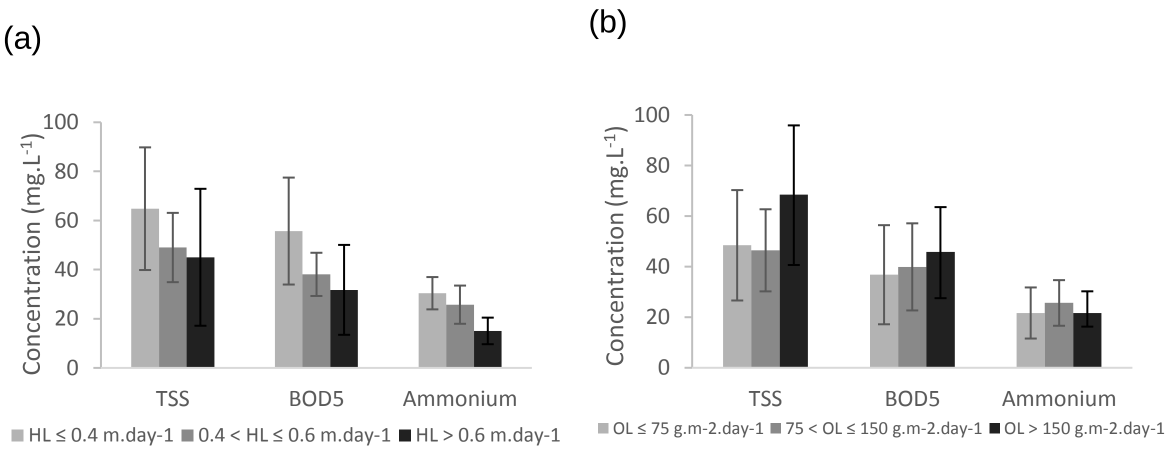


Figure 2-10: Impact of (a) HL and (b) OL on performance of the first stage filters

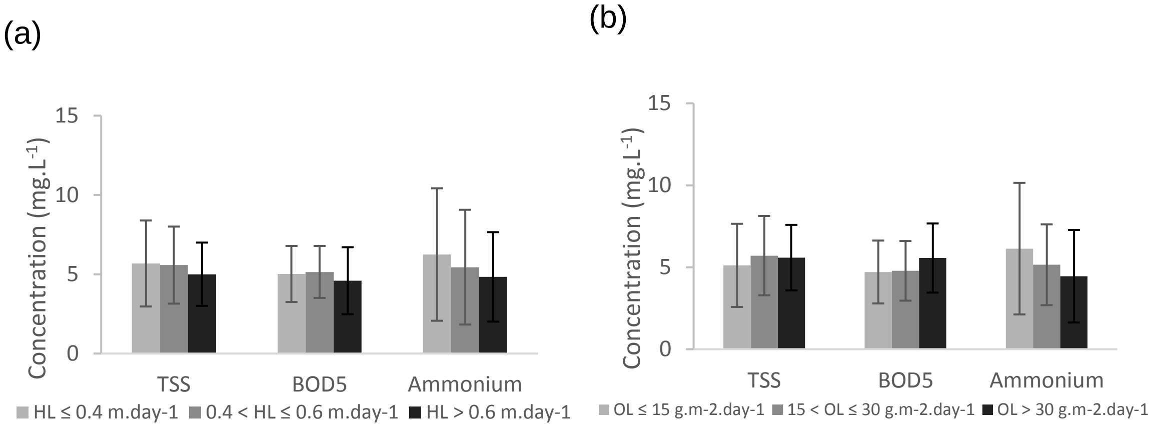


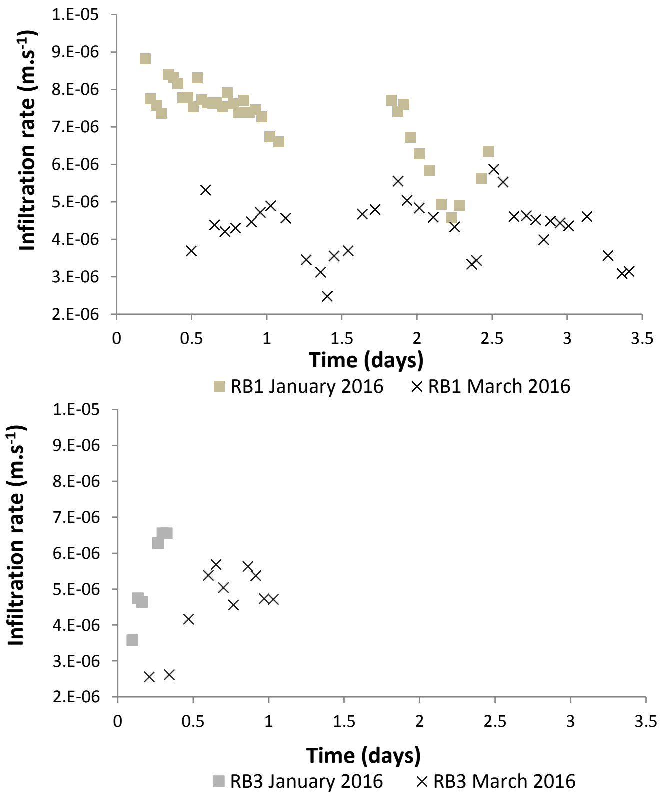
Figure 2-11: Impact of (a) HL and (b) OL on performance of the second stage filters

2.4.2 Infiltration rates and hydraulic behaviour

The infiltration rate varied between 6.6E-06 and 8.8E-06 m·s^{-1} during feeding period one and 2.5E-06 and 5.9E-06 m·s^{-1} for feed period 2 for the filter one (Figure 2-12 (a)) and between 3.6E-06 and 6.6E-06 m·s^{-1} and 2.6E-06 and 5.7E-06 m·s^{-1} for filter three during the first and second trials (Figure 2-12 (b)). This compares to typical level of > 3E-5 m·s^{-1} for mature filters (Molle et al. 2006) and 1E-5 m·s^{-1} for a filter after one year of operation (Arias López 2013). Previous reported profiles describe a decrease in IR during the feeding period as a result of an increasing moisture content in the sludge layer, with the IR stabilizing during the last days of the feeding once the characteristics of the sludge layer have reached a steady state (Molle et al. 2006). This was only observed during the first trial on filter one and was not observed during the other three trials. The difference is attributed to moisture content which was 92% after resting compared to expected levels of 69.8% (Molle et al. 2006).

The IRs observed in filter three initially increased in line with changes in the ponding height until the overflow limit was reached (Figure 2-13(c)). The observed link between IR and ponding height was less evident for the first day of feeding in filter one (Figure 2-13 (a) and (b)). The differences between the filters is considered to be linked to more rapid moisture saturation due to higher hydraulic loading and the poorer reed development restricting natural channelling due to reed oscillation in filter three (Molle et al. 2006). However, the trials were conducted in winter when the impact is reduced as the reeds are dormant. Ponding height remained as the most influencing factor in IR after the first day of operation (Figure 2-13 (a) and (b)) consistent with the importance of sludge layer characteristics on the overall hydraulics. The implication of the impact of ponding height is that draining time can be managed by altering batch volume using bigger batch volumes and less frequent batches as a way to manage high HL in storm periods (Molle et al. 2006). This has to be balanced against a commensurate reduction in residence time during feeding which could negatively affect pollutant removal (Molle et al. 2006).

(a)



(b)

Figure 2-12 Evolution of IR with time for RB 1 (a) and RB 3 (b) during two different months (January and March)

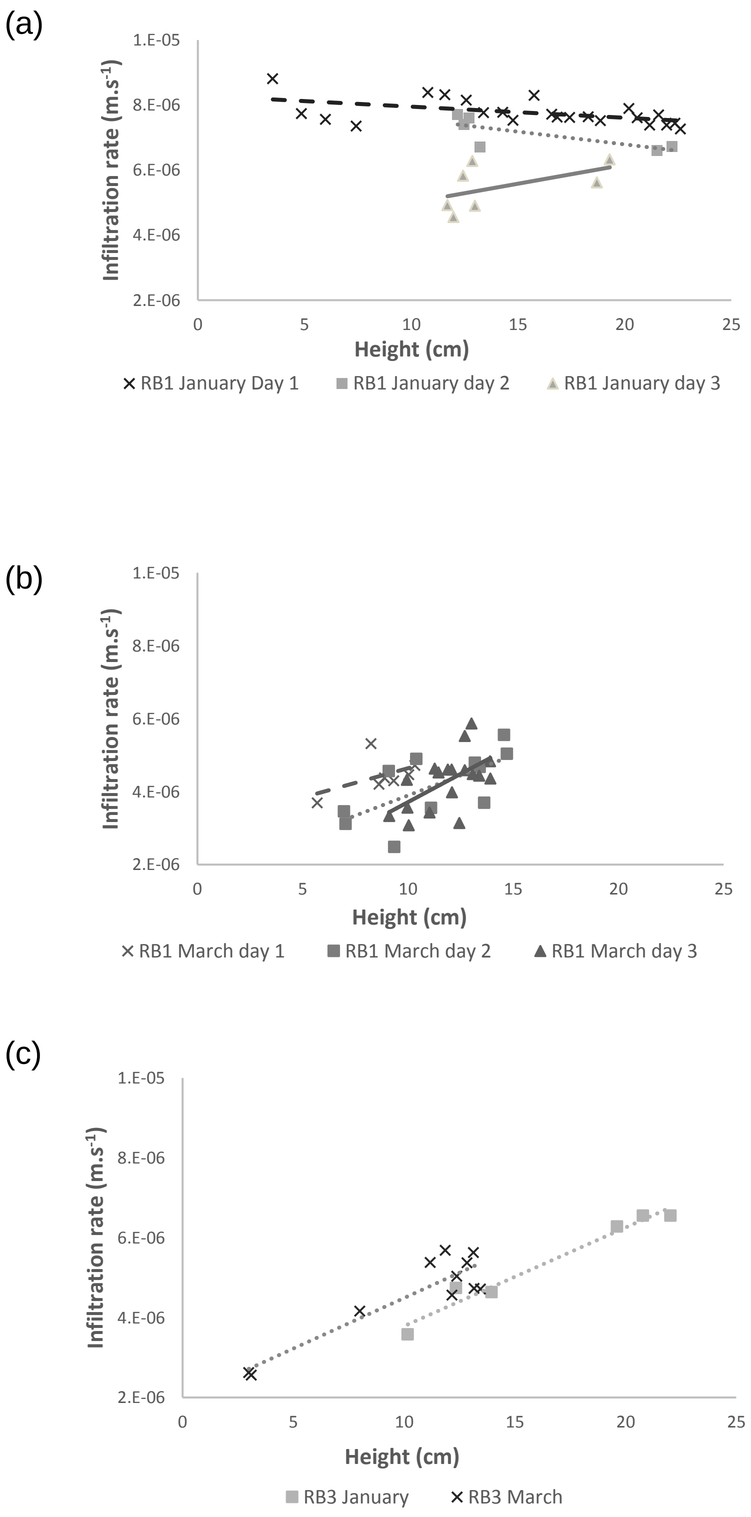


Figure 2-13 Evolution of IR with ponding height for RB 1 (a) (b) and RB 3 (c) during two different months (January and March)

2.4.2.1 Influence of HL distribution on IR, ponding time and re-oxygenation.

The HL distribution of the current site was compared to an equivalent site in France (site A) with a similar average HL of 0.6 m·d^{-1} (Arias López 2013) (Figure 2-14). The current site can be characterised as being exposed to more frequent but less intense HL events, compared to less frequent but more intense HL events registered in site A (Arias López 2013). To illustrate, in site A in France the filter was exposed to HL close to the design value (0.37 m·d^{-1}) more than 50% of the time with occasional episodic HL overloads of 1.2 to more than 2 m·d^{-1}, 15% of the time. In contrast, in the current site the HL were more spread with values between 0.4 to 1.2 m·d^{-1} 63% of the time and less episodic HL overloads of 1.2 to more than 2 m·d^{-1}, accounting for only 6% of the time. Previous experience suggests the filters are able to cope with episodic HL of 1.8 m·d^{-1} once a week and of 3.5 m·d^{-1} once a month, as long as it is exposed to design values during a high percentage of the rest of the time in order to recover the natural dynamics of re-oxygenation and allow the sludge mineralization (Molle et al. 2006). Accordingly, the HL pattern in the current site generates prolonged periods of stress, inhibiting the stabilisation of the systems.

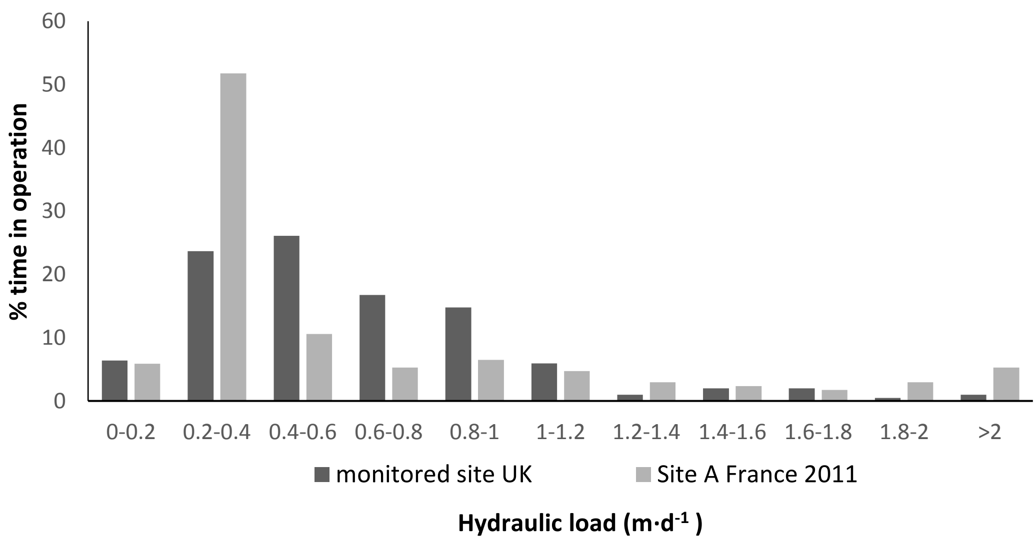


Figure 2-14 Distribution of different HL events over time (July to January) recorded in the monitored site compared to site A (Arias López 2013) in France

The implications relate to ponding periods and the ability of the system to appropriately re-oxygenate so that the biofilm can process the adsorbed load. Previous recommendations have indicated that the filter surface has to stay water free for at least 12 hours per day (Molle et al. 2006) with a more recent study suggesting a maximal cumulative ponding time of 15.5 h per day without causing treatment problems (Arias López 2013). The impact for the current case would be that at the observed average IR of 5.63E-06 $m \cdot s^{-1}$, the daily HL would need to be 0.31 $m \cdot d^{-1}$, a 16% reduction from traditional design value of 0.37 $m \cdot d^{-1}$. This highlights the hydraulic limitations of the young filter (5 months of operation) especially in winter conditions.

2.4.3 Sludge layer characteristics

Characteristics of the sludge layer revealed DM levels of 8.1 ± 1.4%, and 12.8 ± 1.9% for filter one in months five and eight respectively (Table 2-3). Comparative data for filter three was consistent with an increased during operation from 6.5 ± 0.6% in month three to 11.5 ± 01.9% DM in month six. Corresponding OM levels were 75.9 ± 8% and 67.4 ± 7.8% for filter one and 74 ± 12.6% and 74 ± 12.6% for filter three. In Comparison, the average OM of the raw sewage is 85% of the TSS indicating an OM reduction of 12% for the first 3-6 months of operation and 22% for 6-9 months of operation. Previous studies in similar flowsheets in France (Table 2-3) report lower OM content in the sludge layer accumulated in the first stage of a two stage VFCW at between 37.6 to 54% OM. Further results are reported for sludge treatment wetlands where 40% to 50% OM content can be reached with a corresponding a reduction of OM from 25 to 30% (Uggetti et al. 2010). In terms of sludge dewatering Uggetti et al. (2010) reports that DM content of 20-30% can be achieved in sludge drying beds which is similar to those reported for VFCWs at between 15.4 to 30.2% (Table 2-3), in general higher values than the ones found in the sludge in the current case.

Sludge accumulation rates of 3 to 5 cm in 9 months were observed compared to reported levels of between 1.5-2.5 cm a year in mature systems (Molle 2005; Paing et al. 2015; Molle 2014). The Average TSS mass load in the first stage for the system was 143.4 ± 96 $g \cdot m^{-2} \cdot day^{-1}$ in line with proposed design rates 150

$g \cdot m^{-2} \cdot day^{-1}$ suggesting that TSS overload is not the cause for the higher incoming load. The implication is that the sludge layer is not operating at a sufficient mineralisation rate during the initials stages. Comparing IRs found in the current system to IRs found in other similar flowsheets reported with thicker sludge layers (Table 2-3) suggests that the extent of mineralization (Figure 2-15) limits filter percolation to a greater extent than sludge layer thickness (Molle et al. 2006).

(a) (b) (c)

Figure 2-15 State of the sludge layer in August (a) (b) (cracking), and in September (c) (plastic state)

Table 2-3 Sludge layer characteristics measured in the system in terms of accumulation, DM and VS and compared to literature values

		Time in operation	Sludge accumulation (cm)	Accumulation rate ($cm \cdot year^{-1}$)	DM (%)	OM as VS (% of DM)	IR $m \cdot s^{-1}$
December 2015-January 2016	F1	5-6 months	2-5 cm		8.1±1.4	75.9±8	<8.8E-6
	F3	3 months	2-3 cm		6.5±0.6	74±12.6	<6.5E-6
March 2016	F1	8 months	3-5 cm	4-7	12.8±1.9	67.4±7.8	<5.9E-6
	F3	6 months	3-5 cm	(estimated)	11.5±1.9	64.5±7.3	<5.7E-6
Literature	(Arias López 2013)						
	SITE A France	1 year					≈1E-5 (winter)
	(Molle et al. 2006)						
	SITE B France	8 years	4-7 cm	0.4-0.8	30.2	37.6	>5E-5
	(Molle 2005)				(mixed sludge) 21.8	(mixed sludge) 49.2	>3E-5
	(Molle et al. 2006)				(top layer) 15.4	(top layer) 54	
	(Boutin and Liénard 2003)				(middle layer) 21.3	(middle layer) 48.3	
	SITE C France	14 years	22.5 cm	1.6	(lower layer) 26.4	(lower layer) 41.5	
	(Molle et al. 2006)						
	SITE D France	1 year	1 cm	1			>3.6E-4
	(Uggetti et al. 2010)						
	Sludge drying beds				20-30%	40-50%	

2.5 Conclusion

Overall, the site performed similar to values reported in literature regarding total suspended solids, BOD_5 and NH_4^+-N being respectively 6.2 ± 3.4 $mg \cdot L^{-1}$, 5.6 ± 2.6 $mg \cdot L^{-1}$ and 5.8 ± 3.8 $mg \cdot L^{-1}$ compared to literature values of 10 ± 10 $mg \cdot L^{-1}$, 6 ± 4 $mg \cdot L^{-1}$ and 5 ± 6 $mg \cdot L^{-1}$. A key difference compared to operating systems in France was sustained operating periods beyond the design hydraulic load leading to long periods of surface ponding. This had two major impacts: a limiting ability to re-oxygenate the filter body affecting the nitrification performance and retardation of the sludge mineralisation rate reducing the operating infiltration rate and hydraulics of the filters. This highlights the hydraulic limitations of the young filter (5 months of operation) especially in winter conditions.

The implications for further use of the technology are a need to ensure appropriate adaption linked to the actual hydraulic loading profile observed at the target site. The results observed in the current study suggest that appropriate adaptions include: extending the first stage filter area, optimising feeding strategy, using an additional storm and first stage overflow CW to ensure the systems operates within the specified maximum ponding duration. In addition, better understanding of how the sludge layer characteristics evolve will lead to further adaptions to the technology that will accelerate the maturation for the sludge layer enabling stable operation in time period less that the two years currently stated.

3 OVERALL DISCUSSION

Traditionally, the process of implementing technologies that are novel to the UK but established in other countries involves the use of a prototype or pilot in an attempt to understand adaption to local conditions. However, such approaches slow down implementation and so there is an increasing desire to fast track technologies proven elsewhere as in the case of the current study. The observed results and investigation highlighted a number of areas for consideration to manage potential performance risks that may require adaption from the established design.

In the current case the key observed risk was based on the high cumulative ponding heights recorded that negatively influenced several aspects of the dynamics of the filter:

- Poor re-oxygenation leading to lower treatment performance.
- High water content and slow mineralization rates in the sludge layer reduced IR limiting the effective hydraulic acceptance of the filters.

In order to understand the re-oxygenation process in the filters, dissolved oxygen (DO) was measured in the effluent after the second stage, within the discharge arm. However no clear correlation between nitrification and dissolved oxygen at this measuring point was found (Figure 3-1). Expectations were that higher nitrification would occur when DO readings are higher, suggesting that oxygen availability is not a limitation. In contrast, DO varied in a better relation with flows (Figure 3-2). The implication is that use of bulk DO measurement in the effluent is a poor surrogate for actual DO levels in the accumulated biomass. Practical limitations restrict more sophisticated approaches (i.e. micro electrode probes into biofilms) on full scale sites such that understanding oxygenation into biofilms will need to be investigated at a small scale to enable controlled measurement directly into the biomass and then scaled up to full scale operation. Further, current understanding suggests that the majority of the utilised DO originates from diffusion from the gas filled pores during periods of drained operation.

Accordingly, the oxygen available in the free pores can be suggested to be a more representative way to analyse re-oxygenation in the bed. Operationally this will be linked to the resting/feeding periods and the porosity of the media. As such, in full scale situations measurement of the time the bed is drained, by use of level probes, offers a potential means to understand if effective oxygenation is likely to be provided.

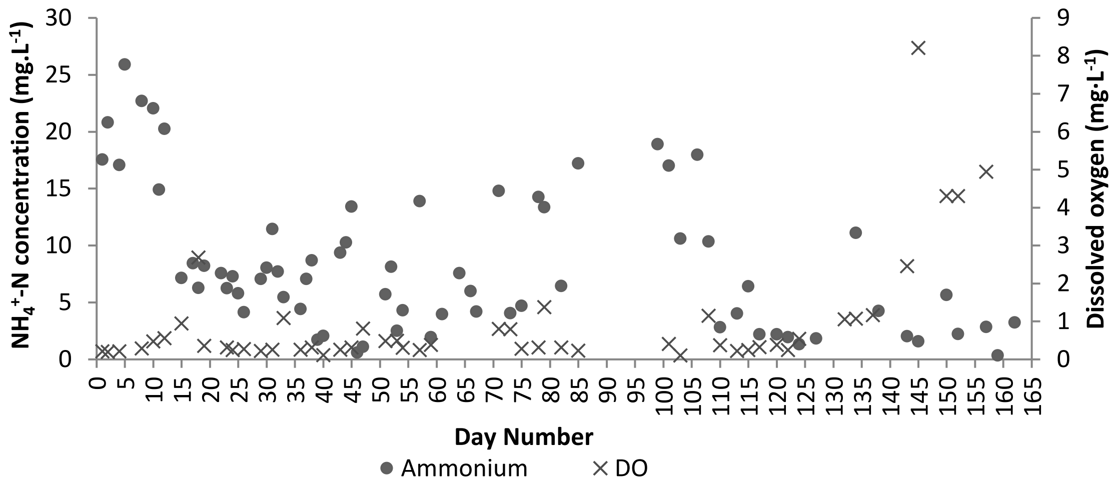


Figure 3-1 Evolution of spot sampling ammonium and dissolved oxygen after second stage filters of a two stages VFCWs system

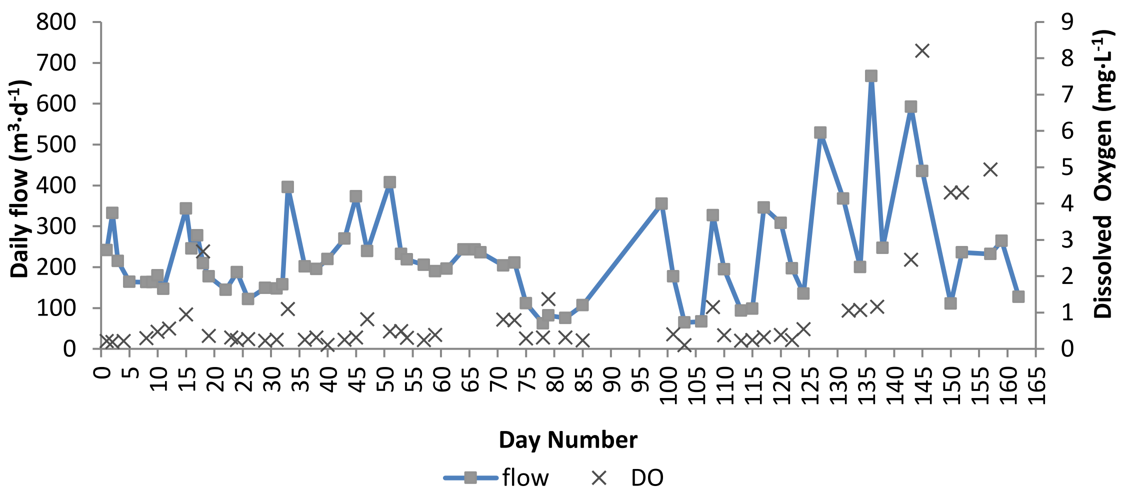


Figure 3-2 Evolution of daily flow and spot sampling dissolved oxygen after second stage filters of a two stages VFCW system

Higher sludge accumulation rates and OM contents (measured as VS) and lower DM were observed in the current study compared to previously reported systems, suggesting that mineralisation occurred at a slower rate. However, the scarcity of literature available for young filters (less than 6 months of operation) prevents a clear understanding of whether the lower mineralisation rates observed in this study are typical during process start-up. The observed sludge characteristics are likely to be limiting the IR in the system and hence the overall hydraulic load that can be effectively treated. Further the accelerated development of the height of the sludge layer observed in the current study would result in a reduction in desludge frequency, which is a major beneficial factor in the selection of the technology.

Poorly mineralised sludges are known to present lower permeability due to the higher capillary water retention capacity of OM (Molle 2014). To explore potential causes of the reduced mineralisation a detailed comparison of the current design and operating practice was compared to the standard practice in France (Table 3-1). Analysis revealed the most likely reason to be associated to differences in the hydraulic load profile. The current system operated close to the design value only 30% of the time compared to 55% for the comparative system in France even though the average hydraulic load was the same for both systems. The periods of excess hydraulic load increased the time the system remained ponded inhibiting sludge mineralisation kinetics reducing IR and hence inhibited the maturation of the sludge layer characteristics. Current recommendations suggest that a minimum of 8.5 hours of free water surface a day needs to be achieved in order to maintain proper oxygenation through the filter surface without affecting filter performance (Arias Lopez, 2013). Measurements in the current case showed periods were the ponding lasted more than 48 hours. Sludge layers can be improved with long resting periods but overall this has to be achievable across the whole system.

Operating strategies are also reported to require adaption to local climatic conditions in terms of rainfall pattern as filters are more sensitive to periodicity rather than intensity of rain events. Additionally the inhibitory impact of low temperatures on kinetics indicate that the duration and severity of winter conditions needs to be reflected in the design. However, current understanding does not specify the specific limiting conditions as the link between IR and sludge layer characteristics is poorly understood.

Analysis of other features related to the design and operation of the current system compared to installations in France found no specific variation in relation to media properties, the definition of population equivalent or typical sewage characteristics in terms of COD/BOD_5 and NH_4^+-N/COD ratios (Table 3-1). Accordingly the recommended organic load levels of 150, 300 and 25-35 $g{\cdot}m^{-2}{\cdot}day^{-1}$ of TSS, BOD_5 and TKN and the associated area per p.e. do not require adaption for UK situation. However, one specific difference was in the level of screening. In the current case no screening was provided to maximise the low maintenance aspects of the technology. In comparison, typical systems in France utilise a 2-3 cm screen to remove rag solids (Paing et al. 2015; Troesch et al. 2014). Visual inspection during the sludge sampling campaign did not indicate an issues with ragging on the filters. However, given the issues discussed above the impact of not screening requires further consideration once the hydraulic load issues are resolved.

In addition the discussed technological aspects, a further important difference was encountered in relation to water quality reporting. UK legislation bases its consent limits on spot sampling whereas the majority of reported data is based on 24 hour composite samples. The importance of this is that observed distribution based on spot values tends to be greater than the commensurate ranged reported with 24 hour composites. The reason is that composites samples are buffered by low values during the night or periods with less activity whereas some of the spot samples occur during specific times of the day when wastewater concentration in the inlet is stronger.

The impact of higher variation with spot samples relates to the interpretation of the robustness of treatment (Figure 3-3 to 3-5). For TSS and BOD_5 spot values suggest a 95%ile value of 20 $mg{\cdot}L^{-1}$ and 15 $mg{\cdot}L^{-1}$ respectively compared to 9.3 $mg{\cdot}L^{-1}$ and 7.7 $mg{\cdot}L^{-1}$ when using composites. This is most pronounced for NH_4^+-

N with a 95%ile value of 16.8 mg.L^{-1} with 4 out of the 64 samples exceeding the target. In contrast, with composites the 95%ile value was 11.9 mg.L^{-1} with all values below the target. The importance to ammonia reflects the inherent batch cycle that the process operates by with expected peaks in the final stages before a rest cycle (section 2.4.1). A suggested strategy is changing the filter rotation in the second stage from 1 week to a shorter period of 3.5 days. This could avoid reaching a higher state of ammonium saturation in the biofilm and hence higher peak values. This is provided a good aeration in the filter body is maintained.

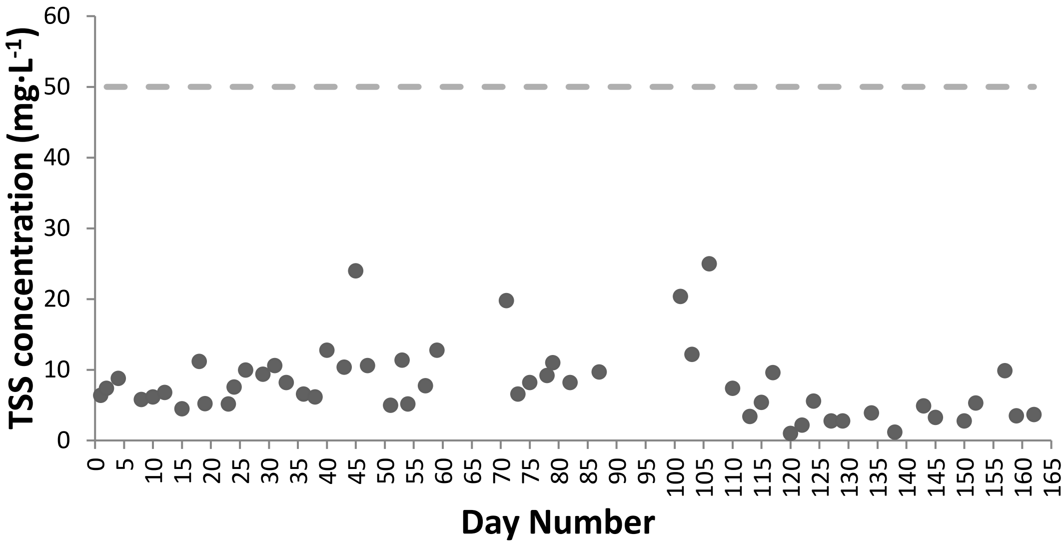


Figure 3-3 Evolution of TSS spot samples over time in the effluent of second stage of a two stages VFCW system

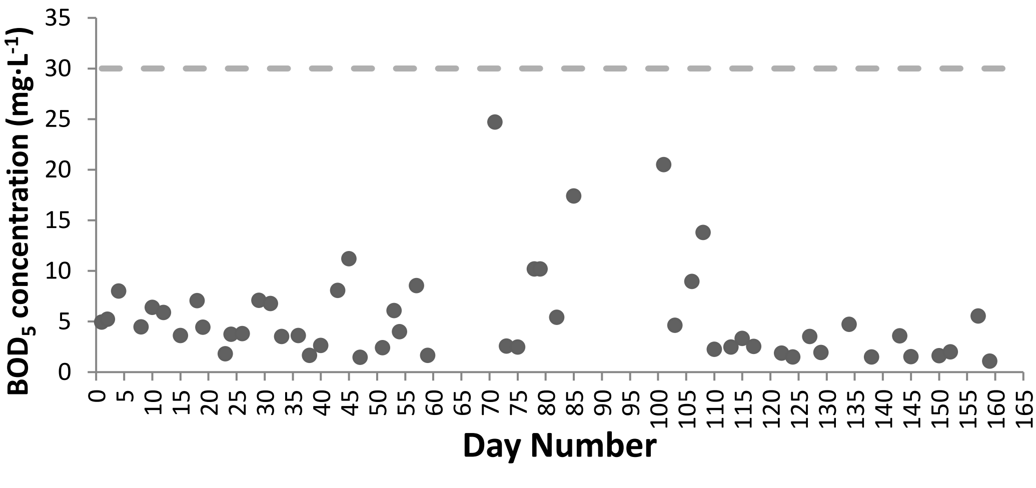


Figure 3-4 Evolution of BOD_5 spot samples over time in the effluent of second stage of a two stages VFCW system

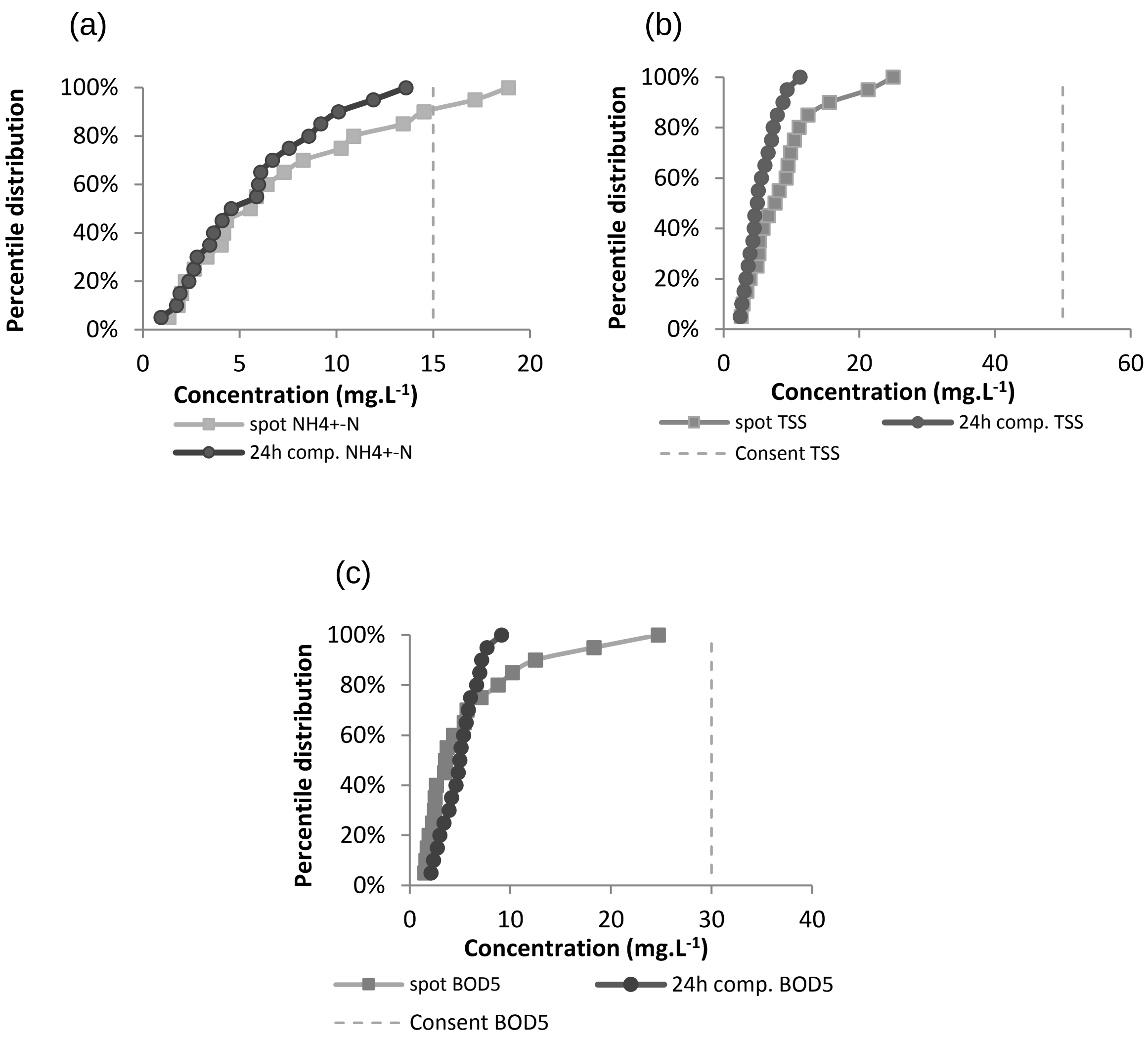


Figure 3-5 Percentile distribution of NH_4^+-N (a), TSS (b) and BOD_5 (c) spot samples in the effluent of the second stage of a two stages VFCW compared to 24 hour composite samples.

Table 3-1 Technology transfer analysis

Design feature	ST design	French design	Reality	Hypothesis / questions	Lessons learnt/adaptations
P.E.	941 (1 PE represents, for 1 day: 160L with 60g BOD_5, 60g SS, 115g COD, 10g TN and 2g TP)	(1 PE represents, for 1 day: 150L with 60g BOD_5, 90g SS for combined sewer , 60g SS for separate sewers, 120g COD, 15g TN and 4g TP) (Troesch et al. 2014) (1 PE represents, for 1 day: 150L with 60g SS, 120g COD, 1-12g TKN) (Molle et al. 2005)		p.e.definition is in line	
HL **($m.d^{-1}$)** - **DWF** - **Average**	2.27 $L{\cdot}s^{-1}$ (0.52 $m{\cdot}d^{-1}$) 1.87 $L{\cdot}s^{-1}$ (0.43 $m{\cdot}d^{-1}$)	0.37 $m{\cdot}d^{-1}$ 0.6 $m{\cdot}d^{-1}$ (Arias Lopez 2013)	0.65 $m{\cdot}d^{-1}$	Were the HL applied higher than design values?	Averages are similar but HL distribution is key factor for a proper re-oxygenation.

Load **(g·m^{-2}·day^{-1})** - - **TSS** - **COD** - **TKN**		150 300 25-35 (Morvannou et al. 2015)	143.6 193.9 18.3	The organic loads are in accordance with design values.	
Area **(m^2·p.e.$^{-1}$)** **Total** **First stage** **Second Stage**	2 1.2 0.8	2 1.2-1.3 0.7-0.8 (Molle et al. 2015)	2 1.2 0.8	The definition of PE in UK is valid for this design parameter of area per p.e., having applied similar pollutant loads.	
Media - **1st stage** - **2nd stage**	40 cm gravel 2/6mm 20-40 cm 10/63 mm 40 cm sand 0/4 mm 40 cm gravel 2/6mm 20cmgravel20/40mm 0-30cmgravel10/63mm	Variable, most common 40–50 cm gravel 2/8 mm 15–20 cm gravel 10/20 mm 20 cm gravel 20/40 mm (Paing et al 2015) 40 cm sand 0/4 mm 15–20 cm gravel 4/10 or 4/20mm 20 cm gravel 10/20 or 20/40 mm (Paing el al. 2015)		The gravel/sand shows standard distribution	

Feeding system - **1st stage** - **2nd stage**	No screening Pumping station Siphon	Screening Siphon/pumping station Siphon/pumping station		Absence of screening had any effect in sludge accumulation?	
Feeding points - **1st stage** - **2nd stage**		1 per max 50m^2 (Troesch et al. 2014) 1 per 1m^2 (Troesch et al. 2014)	1 per 47m^2 1 per 1m^2		
Feeding/resting period - **1st stage** - **2nd stage**	3.5 days/1 week 1week/1week	Variable, most common: 3.5 days/1 week (Morvannou et al. 2015) (Troesch et al. 2014) 1 week/2 weeks (Paing et al. 2015) 1week/1week (Paing et al. 2015) 3.5 days/3.5 days (Morvannou et al. 2015)	2.5-3.5/5-7 5-7/5-7	Feeding strategy. Have using shorter feeding periods influenced mineralization sludge layer?	

Volume per batch (m^3) - **1st stage**	7.5 (2 cm over surface)	3-4 cm over surface (Paing et al. 2015) 1.3 cm (Molle 2014) 3cm (Paing et al. 2005)	Variable	Batch management. Has batch management influenced IR and sludge mineralization?	
- **2nd stage**	7.5	4cm (Paing et al. 2005)	Syphoning malfunction		
Sludge accumulation - **cm accum. (cm·year^{-1})**		1.5 (Molle et al. 2005) 2.5 (Molle 2014) 1-2 (Troesch et al. 2014)	2-5 cm/5 months	Was mineralization rate not optimal compared to other similar sites in the first year of operation? Can mineralization rate be improved by proper hydraulic management?	It is necessary to promote the sludge mineralization by managing HL, batches and feeding/resting periods.
- **Dry mater (%)**		30.2 (Molle et al. 206)	6.7		
Organic matter (%)		37.6 (Molle el al. 2006)	74.3		

Reed type - **Type** - **Density**	Phragmites australis 4 per m^2	Phragmites australis 1 every 50 cm in each direction (Boutin and Liénard 2003)	Poor development of reeds specially in bed n3 and 2nd stage	Is the poor reed development having an impact in IR	
Sewage characteristics - **COD/BOD_5** - **NH_4^+ /COD**		2.66 (min 1.87, max 3.66) 0.07 (min 0.03, max 0.15) (Morvannou et al. 2015)	2.62 (min1.83, max3.93) 0.09 (min 0.03, max 0.17)	The characteristics of the sewage are in the same line	

3.1 Suggested adaptations

Consideration of the observed challenges with implementation in the UK leads to a number of suggested adaptations which should alleviate the situation and increase the efficacy of the technology.

(1) Surface Area.

To adjust the distribution of hydraulic loadings the area of the first stage filter should be based on achieving a 50% operation at the design rate. In the current case this would represent and increase to 1.7 $m^2 \cdot p.e.^{-1}$ (Figure 3-6).

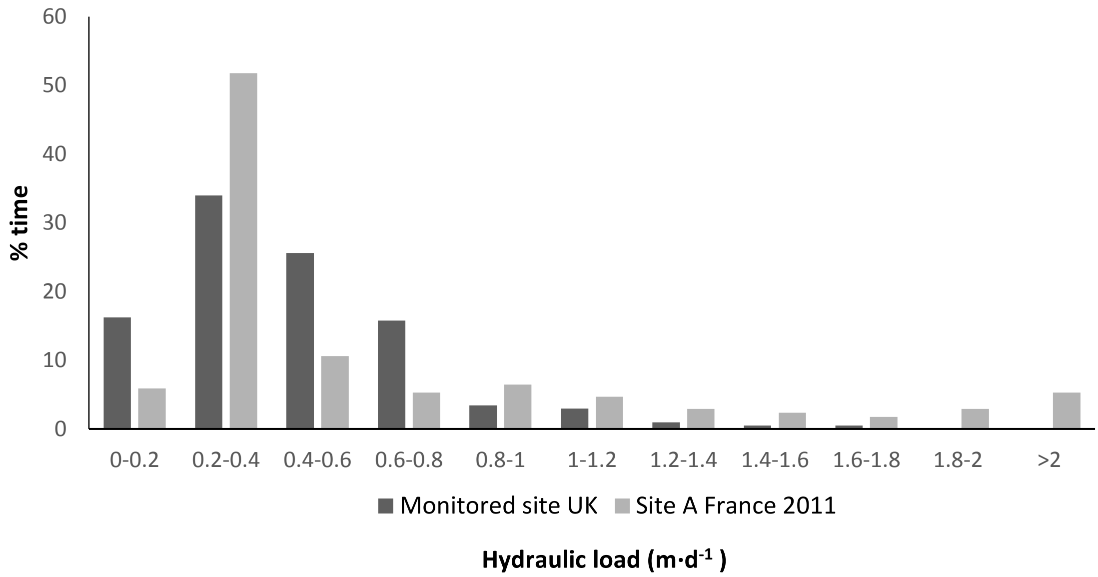


Figure 3-6 Distribution of different HL events over time (July to January) considering a surface area design value of 1.7 $m^2 \cdot p.e.^{-1}$ in the monitored site compared to site A (Arias López 2013) in France

(2) Batch management: bigger batch volumes, less frequent.

During storm periods or hydraulic overload periods a batch management using less frequent and bigger volumes batches should improve filter performance. This will lead to improved IRs, longer resting periods between batches and lower residual water content improving re-oxygenation and sludge mineralisation (Molle 2006). Opposite to this, contact time is shortened negatively affecting pollutant removal. Molle (2006) suggests that batch volume can be doubled without compromising removal efficiency.

(3) First stage rotation (Dry period: 3.5 days rotation / storm period: rotation at a maximum daily HL).

During storm events it is possible to change the rotation strategy to a maximum daily HL instead of a fixed number of days. The filter can function in a classical rotation period for dry weather (3.5 days) or depending on a maximum daily HL for storm periods, changing the filter rotation when reaching a critical daily HL (Arias López 2013).This will lead to a more frequent bed rotation, protecting the filters against long ponding periods, as long as a good recovery is observed during shorter resting periods (Arias López 2013). Establishing a maximum daily HL in order to avoid long periods of ponding depends on the observed IR, which varies with season and system age. Arias López (2013) suggests a value of 100 batches (1.81 m) per day as a maximum HL before changing duty. However, adapting the number of batches per day using the IR observed in the monitored filters considerably reduces the value, due to the low IR registered. Considering an average IR of 5.63E-06 $m \cdot s^{-1}$ and a maximum undesirable situation of 24 hours of continuous ponding gives a maximum daily HL of 24 batches before bed rotation. This value can be adapted depending on IR progress, changing targets for the different seasons and system age.

4 CONCLUSIONS

The overall conclusion of the work is that two stage VFCW appear to provide the potential for effective treatment of sewage at small works. The efficacy of treatment was similar to that observed in established systems demonstrating that a very low energy and maintenance technology can provide appropriate treatment. However a number of areas of difference have been observed that suggest adaption is required for successful implementation in the UK.

The key issue is related to the development of the sludge layer so that the characteristics are sufficient to provide the required infiltration rate. Sufficient time between ponding periods is required to enable sufficient aerobic mineralisation of the sludge layer. The key factor observed in the current case was the periodicity of the hydraulic rates above the design level which prevented sufficient non ponded periods to exist. This impacted on the sludge layer characteristics and consequently the treatable hydraulic load.

Previous experience from France was based on 24 hour composites which buffered variation in comparison to the use of spot sampling. This requires re-evaluation of the effective treatment level achievable with the technology. This need to be resolved once the filter are operating under stabilised conditions.

The main target is achieving a good oxygenation in the filter. This can be achieved by using the correct operational (batch management and feeding/resting periods) and design procedures in line with the HL distribution observed. In the current case the difference in hydraulic profile suggested a number of possible adaptions:

- Increase surface area to 1.7 $m^2 \cdot p.e.^{-1}$.
- Batch management during storm periods: bigger volumes, less frequent.
- 1st stage rotation: 3.5 days rotation for dry periods or rotation at a maximum daily HL during storm events.

5 FUTURE WORK

In order to address the limitations found during this project, the following design and process suggestions have been identified as options to overcome those limitations and adapt the technology to UK context:

- Extend first stage filters in order to achieve the correct hydraulic pressure in the filters.
- Study the possibility of different media gradation for improving percolation
- Aeration of the filter media creating two fold improvement: improve oxygen availability and for flushing solid accumulation. Aeration could be used in a regular basis or just as a back-up plan when high ponding rates inhibit proper bed re-oxygenation or for periodically flushing solids accumulation. The negative side of this option is the increase demand for energy.
- Storm reed beds for treating storm flows can avoid high ponding time and it also will avoid returning to the process septic storm flows accumulated in storm tanks. Storm reed beds could also treat overflow from the first stage avoiding solids carry over onto the second stage.
- Design based on infiltration rate. An estimation of the area needed considering the flow to be treated, the observed IR and maximum recommended ponding rate, will allow to avoid long periods of ponding.

Seeding/bio-augmentation. The long commissioning period required for achieving the correct characteristics of the sludge layer in terms of mineralization rate and bacterial population makes attractive the possibility of seeding the bed with older sludge.

The impact of the sludge layer in constructed wetlands's IR has been identified as critical to ensure a suitable hydraulic acceptance within the treatment flowsheet. Accordingly, the following research is proposed to elucidate the sludge layer characteristics that enable a sufficient IR and to inform operation regimes (feed and batch management) to guarantee a sufficient hydraulic acceptance and treatment performance. Operation of laboratory tests in columns filled with media is proposed, with the objectives to:

- Mimic sludge layers with different characteristics in terms of DM and OM and quantify the system's IR.
- Monitor the progression of the sludge layer in terms of DM and OM or different batch management strategies and feeding/resting periods in order to understand how to promote the most favourable conditions found in the previous point.

It is proposed to complement the above laboratory work with an extensive sludge layer sampling campaign in the full scale system, with a special focus on monitoring DM, OM and IR over prolonged operational times and different seasons.

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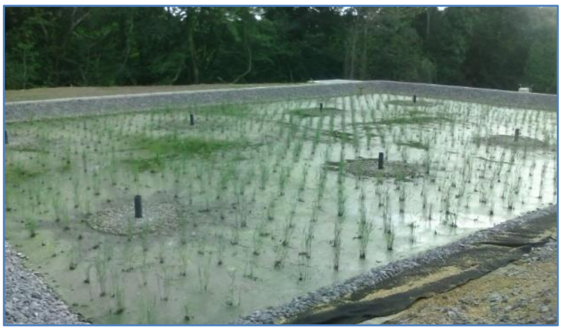
APPENDICES

(a)

(b)



(c)

(d)

(e)

(f)

Figure 1: Filters first stage (a) (b) and second stage (c) (d). Pumping station (e) and syphon chamber (f)

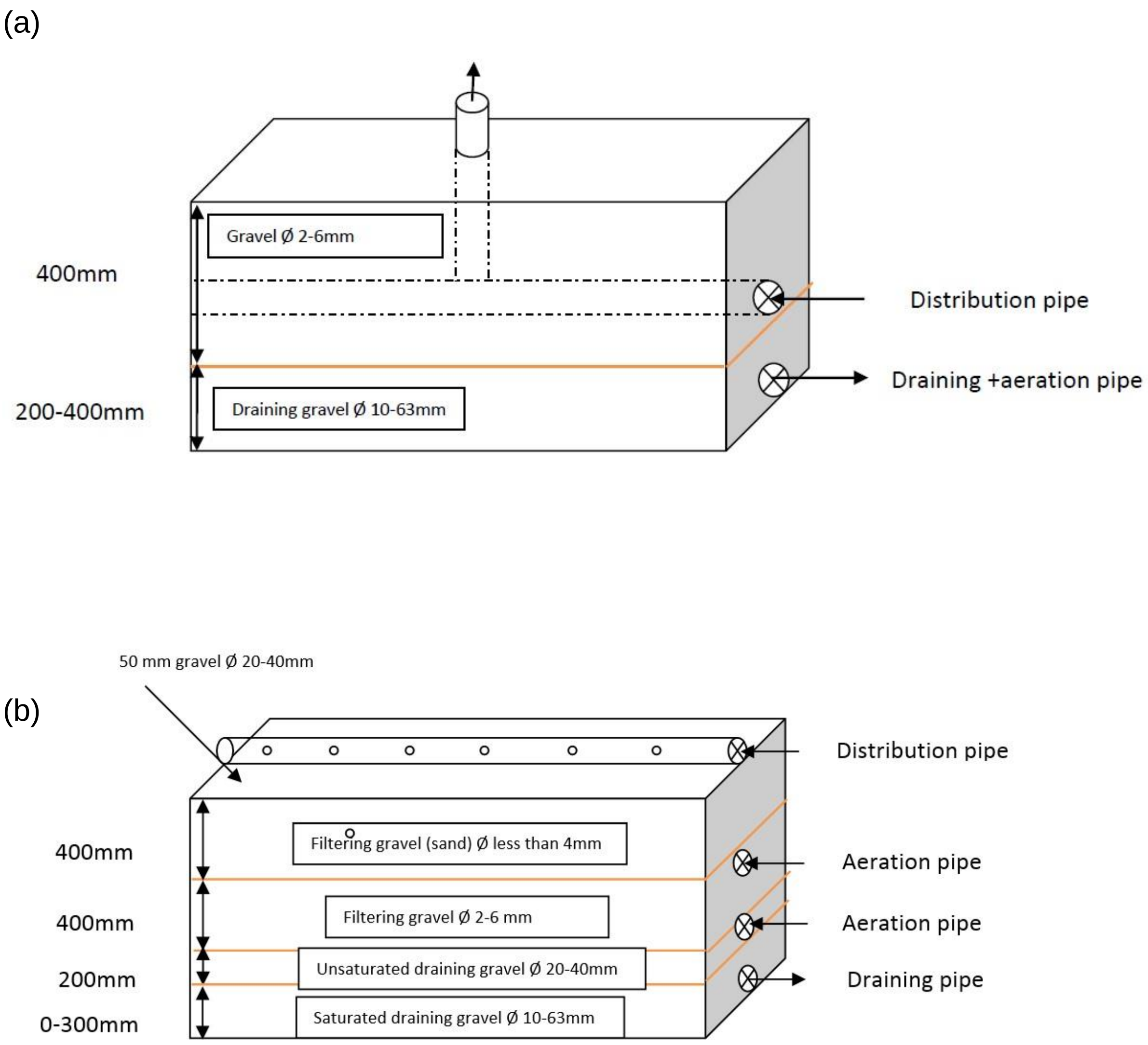


Figure 2: Configuration of the different layers and draining and ventilation pipes in the first (a) and second stage (b)

(a) (b)

Figure 3: Ponding in a filter of first stage (a) and second stage (b)

(a) (b)

Figure 4: Difference in reed development in month five of operation (December) for F1 (a) and F3 (b)

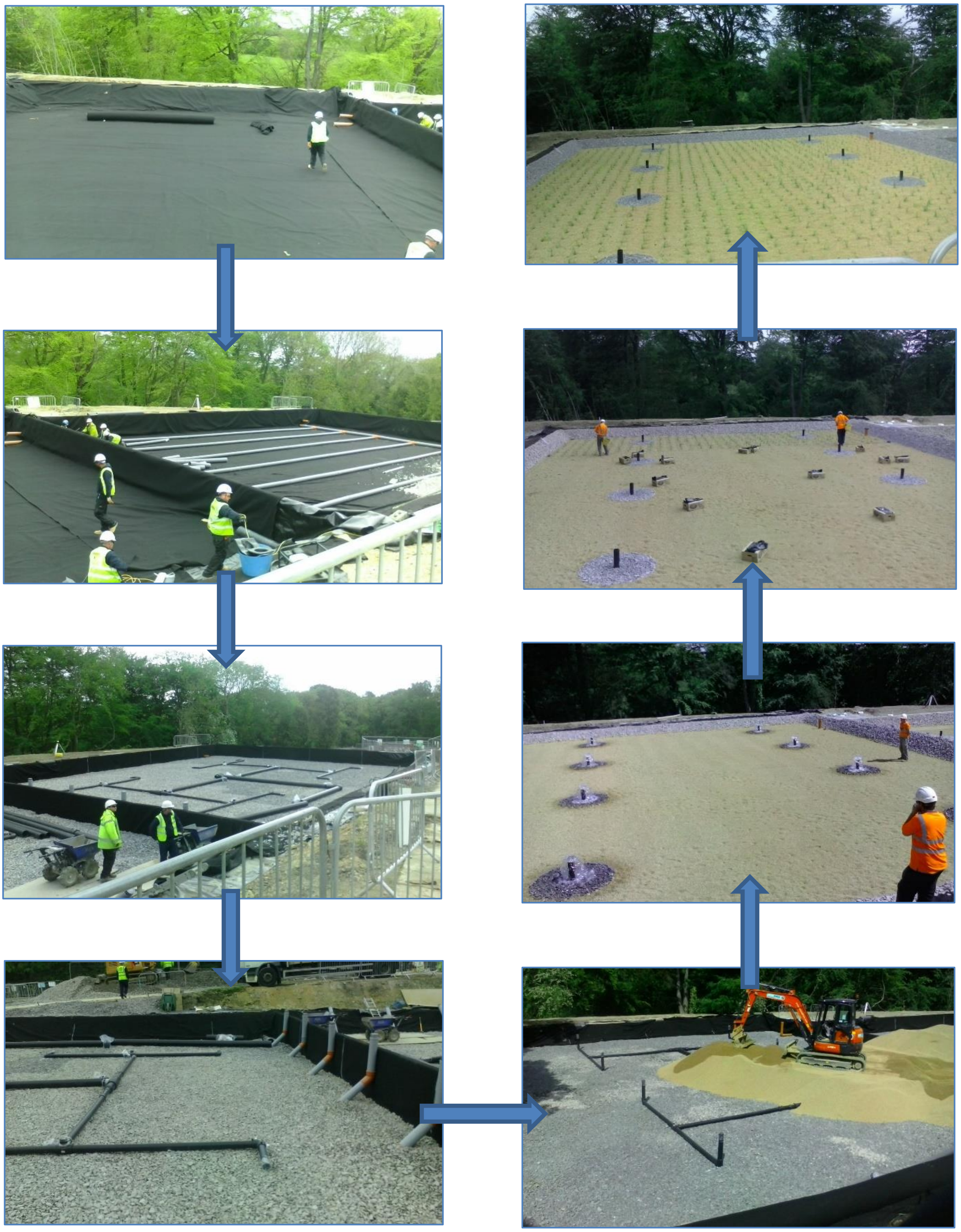


Figure 5: Construction process in a filter in the first stage

Table 1: Metal measurements from July to September 2015

			Mn	Ni	Cu	Zn	As	Cd	Pb	Fe	Hg
Spot sampling			**$\mu \cdot L^{-1}$**	**$\mu \cdot L^{-1}$**	**$\mu \cdot L^{-1}$**	**$\mu \cdot L^{-1}$**	**$\mu \cdot L^{-1}$**	**$\mu \cdot L^{-1}$**	**$\mu \cdot L^{-1}$**	**$mg \cdot L^{-1}$**	**$\mu \cdot L^{-1}$**
	Raw wastewater	Mean	46	5	13.1	77	2.4	0.23	1.2	0.255	0.31
		SD	7.3	0.54	3	22.5	0.5	0.16	0.72	0.053	0.09
		N	7	7	7	7	7	7	7	7	7
	1ststage effluent	Mean	156.2	6.3	8.1	53.7	4.4	0.19	0.64	0.155	0.09
		SD	36.4	0.5	4.5	28.6	0.52	0.09	0.51	0.073	0.07
		N	6	6	6	6	6	6	6	6	6
	2ndstage effluent	Mean	83.1	3.3	8.3	33.5	1.3	0.35	0.36	0.02	0
		SD	42.3	0.72	1.5	7	0.27	0.24	0.17	0.019	0
		N	7	7	7	7	7	7	7	7	7
24 h composite sampling											
	Raw wastewater	Mean	84.9	4.3	12.3	73.6	2.4	0.25	0.88	0.304	0.29
		SD	11.4	0.6	3.4	19	0.4	0.12	0.21	0.018	0.12
		N	3	3	3	2	3	3	3	3	3
	1ststage effluent	Mean	139.6	5.3	6.3	45.9	4.1	0.17	0.41	0.148	0.03
		SD	35.5	0.4	4.5	5.1	0.6	0.08	0.29	0.053	0.06
		N	3	3	3	2	3	3	3	3	3
	2ndstage effluent	Mean	14.4	5.3	6.6	21.1	1.4	0.31	0.41	0.024	0.16
		SD	3.4	0.4	1.2	0.8	0.31	0.11	0.16	0.012	0.206
		N	3	3	3	2	3	3	3	3	3